FREEZE-THAW AND SUSTAINED LOAD DURABILITY OF NEAR SURFACE MOUNTED FRP STRENGTHENED CONCRETE

by

Peter Andrew Mitchell

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Abstract

In recent years, a modified method to strengthen reinforced concrete (RC) structures has emerged involving application of fibre reinforced polymers (FRPs) in the ‘near surface’ of a member. The near surface mounted (NSM) method entails placing a pre-cured FRP bar, rod, strip, or plate, along with an adhesive into a pre-cut groove or slot in the cover of a member. Advantages of the NSM technique over externally bonded (EB) systems include minimal surface preparation and installation time, the ability to anchor the FRP into an adjacent member, superior protection from mechanical and environmental damage, and superior bond properties. Although a number of laboratory studies, field applications, and experimental field projects have employed the NSM FRP strengthening technique, none of these have been performed in a climate where cold environments and freeze-thaw cycling could cause adverse effects. This thesis presents the results of an experimental program to investigate the flexural and bond performance and freeze-thaw durability of a specific NSM carbon/vinylester FRP tape strengthening system through a series of tests on strengthened slab strips and a series of pull-out bond tests. The effects of adhesive type (cementitious or epoxy) and exposure condition (room temperature, freeze-thaw, sustained load, or freeze-thaw under sustained load) are examined. The results indicate no discernable negative impacts on the performance of the grout strengthened members after exposure to freeze-thaw cycles and/or sustained load. The slab strips strengthened with epoxy adhesive displayed minor changes in ultimate load (less than three percent) after exposure to freeze-thaw cycles or a period of sustained load, while the combined effect of freeze-thaw cycles and sustained load produced an average reduction in ultimate load of eight percent. The epoxy adhesive strengthened pull-out bond tests experienced a 27% average drop in ultimate load after 150 freeze-thaw cycles. These results suggest that additional research on the combined effects of
sustained load and freeze-thaw cycling are warranted, particularly for NSM strengthening applications using epoxy adhesives.
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# Table of Contents

Abstract ............................................................................................................................................ ii  
Acknowledgements ......................................................................................................................... iv  
Chapter 1 .......................................................................................................................................... 1  
Introduction ...................................................................................................................................... 1  
1.1 General ................................................................................................................................... 1  
1.2 Need for Rehabilitation .......................................................................................................... 1  
1.3 Fire-Reinforced Polymers ...................................................................................................... 3  
1.4 Evolution of the Near Surface Mounted Technique .............................................................. 3  
1.5 Research Significance ............................................................................................................ 6  
1.6 Research Objectives ............................................................................................................... 6  
1.7 Scope ...................................................................................................................................... 8  
1.8 Outline of Thesis .................................................................................................................... 9  
Chapter 2 Literature Review .......................................................................................................... 13  
2.1 Motivation ............................................................................................................................ 13  
2.2 NSM Background ................................................................................................................ 14  
2.2.1 NSM Rods ..................................................................................................................... 15  
2.2.2 NSM Strips ................................................................................................................... 18  
2.2.3 Field Applications ......................................................................................................... 19  
2.2.4 Bond Failure Modes ...................................................................................................... 20  
2.2.5 Bond Models ................................................................................................................. 23  
2.3 Freeze-Thaw Effects on Structural Materials ...................................................................... 29  
2.3.1 Concrete ........................................................................................................................ 30  
2.3.2 FRPs .............................................................................................................................. 31  
2.4 Freeze-Thaw Effects on FRP Strengthened Concrete Members .......................................... 32  
2.5 Sustained Load ..................................................................................................................... 33  
2.6 Existing NSM Guidelines .................................................................................................... 35  
2.6.1 ACI 440.2R-08 .............................................................................................................. 35  
2.6.2 CSA S806-02 ................................................................................................................ 38  
2.6.3 CSA S6-06 .................................................................................................................... 39  
2.6.4 HB305-2008 .................................................................................................................. 39
2.7 Summary .............................................................................................................................. 40

Chapter 3 Experimental Procedure ........................................................................................ 49
3.1 Introduction.......................................................................................................................... 49
3.2 Slab Strip Specimens ......................................................................................................... 52
   3.2.1 Design ........................................................................................................................... 52
   3.2.2 Fabrication .................................................................................................................... 53
   3.2.3 FRP Strengthening System ........................................................................................... 54
   3.2.4 Test Setup..................................................................................................................... 57
   3.2.5 Instrumentation ............................................................................................................. 58
   3.2.6 Freeze-Thaw Cycling .................................................................................................. 60
   3.2.7 Sustained Load ............................................................................................................ 61
   3.2.8 Testing Procedure ......................................................................................................... 63
3.3 Pull-Out Specimens ............................................................................................................. 63
   3.3.1 Design ........................................................................................................................... 63
   3.3.2 Fabrication .................................................................................................................... 64
   3.3.3 FRP Strengthening System ........................................................................................... 64
   3.3.4 Test Setup..................................................................................................................... 65
   3.3.5 Instrumentation ............................................................................................................. 66
   3.3.6 Testing Procedure ......................................................................................................... 66
3.4 Ancillary Testing ................................................................................................................. 67
3.5 Summary .............................................................................................................................. 67

Chapter 4 Results and Discussion ............................................................................................ 84
4.1 Introduction .......................................................................................................................... 84
4.2 Ancillary Test Results ......................................................................................................... 84
   4.2.1 Concrete ........................................................................................................................ 84
   4.2.2 CFRP ............................................................................................................................. 85
   4.2.3 Reinforcing Steel .......................................................................................................... 86
   4.2.4 Mortar Cubes (Grout Testing) ..................................................................................... 86
4.3 Slab Strip Testing Program ................................................................................................. 87
   4.3.1 Room Temperature Slab Strips ..................................................................................... 87
   4.3.2 Slab Strips Exposed to 300 Freeze-Thaw Cycles ......................................................... 90
   4.3.3 Slab Strips Exposed to Sustained Load at Room Temperature ..................................... 92
List of Figures

Figure 1.1: The aging Rockefeller Road Bridge in Cleveland, Ohio, is one of many decaying RC bridges (ASCE, 2005) .................................................................................................................... 11
Figure 1.2: Constituent materials of FRP (ISIS Canada, 2003) .................................................................................................................... 11
Figure 1.3: Shear strengthening with Externally Bonded (EB) wet lay-up FRP sheets (Rizkalla et al., 2003) ........................................................................................................................................ 12
Figure 1.4: Applying Near Surface Mounted (NSM) FRP rods (Rizkalla et al., 2003) ........................................ 12
Figure 2.1: Modified pull-out test setup designed by De Lorenzis et al. (2002) to investigate the bond performance of NSM FRP rods in concrete (De Lorenzis et al., 2002) ......................... 42
Figure 2.2: Cohesive shear failure in the concrete (left), and tensile splitting of the adhesive (right) observed in modified pull-out bond testing of NSM FRP rods (De Lorenzis et al., 2002) 42
Figure 2.3: Normal bond stress distribution around a recatngular NSM strip (left) and a curcular NSM rod (right) showing how the bond stresses around a rod create forces which push the rod out of the adhesive ............................................................................................................................... 43
Figure 2.4: Installing NSM carbon FRP rods on the soffit of a bridge deck near Rolla, Missouri (Alkhrdaji et al.,1999) ............................................................................................................................................. 43
Figure 2.5: Cutting grooves prior to installation of NSM FRP bars to strengthen a reinforced concrete silo (Prota et al., 2001) ......................................................................................................................................... 44
Figure 2.6: Parameters and dimensions used in the Seracino et al. (2007a) models for bonded rectangular FRP plates and strips. The aspect ratio is the ratio of the depth of the failure plane to the width of the failure plane. Externally bonded plates have a small aspect ratio (left) and near-surface mounted plates have a larger ratio (right) (Seracino et al., 2007a) ............................................................................................................................................. 44
Figure 2.7: non-air-entrained concrete (left), and air-entrained concrete (right) showing uniformly distributed air voids throughout the paste of the concrete with a common pin for reference (http://www.cement.org/tech/cct_dur_freeze-thaw.asp) ............................................................................................................................... 45
Figure 2.8: Summary of FRP rods examined in the durability investigation performed by Micelli (2004) ............................................................................................................................................. 45
Figure 2.9: ASTM D445 test setup used in the durability study by Micelli (2004) ............................................................................................................................... 46
Figure 2.10: Schematic of the environmental exposures used in the durability study by Micelli (2004) ............................................................................................................................................. 46
Figure 2.11: Schematic showing simplified creep and creep recovery for concrete under uniaxial stress (Bisby, 2006)........................................................................................................................................ 47
Figure 2.12: Schematic showing the intermediate crack (IC) induced debonding mechanism for externally bonded FRPs bonded to concrete substrates (Seracino et al., 2007)........................................................................ 47
2.13: Various potential debonding mechanisms for FRP strengthened reinforced concrete beams and slabs (Oehlers, 2005)........................................................................................................................................... 48
Figure 3.1: Slab strip loading frame and test setup........................................................................ 72
Figure 3.2: Elevation (a) and section (b) views of the slab strip dimensions and reinforcement details (after Burke, 2008)............................................................................................................................................... 72
Figure 3.3: Slab strip formwork with steel reinforcement in place prior to casting the concrete (after Burke, 2008).................................................................................................................................................................................... 73
Figure 3.4: Consolidating the fresh concrete in the formwork ...................................................... 73
Figure 3.5: Tuckpoint grinder used to cut grooves for NSM in the concrete cover....................... 74
Figure 3.6: Two views of the aluminum angle grinder guide and slab strip prior to cutting an NSM groove (after Burke, 2008).................................................................................................................................................. 74
Figure 3.7: Cutting NSM groove in slab strip specimen using aluminum jig (after Burke, 2008) 75
Figure 3.8: 100mm gauge length Pi Gauge used in the current study ........................................... 75
Figure 3.9: Caulking and foam barriers used to dam the polymer or grout adhesive in the NSM groove during strengthening installation (after Burke, 2008)............................................................................................................................................................................ 75
Figure 3.10: Thermocouples installed on a thermocouple frame within in slab-strip formwork prior to casting the concrete......................................................................................................................................................................................... 76
Figure 3.11: Unprotected strain gauge (top) and strain gauge protected with marine grade black silicone (bottom) coupled with wiring in sealed black tubing to prevent damage to the strain gauge due to exposure to water in freeze-thaw tank........................................................................................................................................................................................................................................ 77
Figure 3.12: Schematic showing location of loading points and locations of Pi gauges (PI), linear potentiometers (LP) and foil strain gauge (SG) (after Burke, 2008) ................................................................................................................................................................................... 78
Figure 3.13: Freeze-thaw temperature cycles for a typical slab strip specimen during a 24 hour period .............................................................................................................................................................................................. 78
Figure 3.14: Sustained load being applied to a pair of back-to-back slab strip specimens using a hydraulic jack and two load cells........................................................................................................................................................................................................................................ 79
Figure 3.15: Sustained loading setup after load is applied to a pair of back-to-back slab strip specimens........................................................................................................................................................................................................................................ 79
Figure 3.16: Freezing in air (left) and thawing in water (right) of slab strip specimens under sustained load.........................................................................................................................80
Figure 3.17: The C-shaped concrete block designed by De Lorenzis et al (2002) that was used for the modified pull-out testing program. .........................................................................................................................80
Figure 3.18: Plan (a) and elevation (b) views of the direct pull-out specimens used in the current study........................................................................................................................................81
Figure 3.19: Formwork used for the modified pull-out specimens.......................................................................................................................... 81
Figure 3.20: 3-D computer representation and actual pull-out specimen in the test setup in preparation for testing. The aluminum cylinder was only used to support the grips prior to testing and was no longer needed once the hydraulics were running............................................................... 82
Figure 3.21: High resolution digital image taken for pixel tracking of displacements and optical strain measurement during pull-out testing .........................................................................................................................83
Figure 4.1: Peak loads for all slab strips and average loads for each specimen type............. 112
Figure 4.2: Peak FRP strains as a fraction of ultimate strain for each slab strip and average for each specimen type .............................................................................................................................................. 112
Figure 4.3: Total applied load versus midspan deflection for all conditioning regimes for all grout adhesive strengthened slab strips........................................................................................................................................... 113
Figure 4.4: Midspan moment versus midspan curvature for all conditioning regimes for all grout adhesive strengthened slab strips ........................................................................................................................................... 113
Figure 4.5: Midspan moment versus midspan FRP strain for all conditioning regimes for all grout adhesive strengthened slab strips ........................................................................................................................................... 114
Figure 4.6: Total applied load versus midspan deflection for all epoxy adhesive strengthened slab strips. Data taken from Burke (2008) are shown using dashed lines ................................................................. 114
Figure 4.7: Midspan moment versus midspan curvature for all conditioning regimes for all epoxy adhesive strengthened slab strips. Data taken from Burke (2008) are shown using dashed lines ........................................................................................................................................... 115
Figure 4.8: Midspan moment versus midspan FRP strain for all conditioning regimes for the epoxy adhesive strengthened slab strips. Data taken from Burke (2008) are shown using dashed lines ........................................................................................................................................... 115
Figure 4.9: Total applied load versus midspan deflection for unconditioned slab strips tested at room temperature. Data taken from Burke (2008) are shown using dashed lines ................. 116
Figure 4.10: Moment versus curvature at midspan for unconditioned slab strips tested at room temperature. Data taken from Burke (2008) are shown using dashed lines.

Figure 4.11: Midspan moment versus FRP strain in the notch at midspan for unconditioned slab strips tested at room temperature. Data taken from Burke (2008) are shown using dashed lines.

Figure 4.12: Total applied load versus deflection at midspan for freeze-thaw cycled but unloaded slab strips.

Figure 4.13: Moment versus curvature at midspan for freeze-thaw cycled but unloaded slab strips.

Figure 4.14: Moment versus FRP strain in the notch at midspan for freeze-thaw cycled but unloaded slab strips.

Figure 4.15: Total applied load versus deflection at midspan for slab strips conditioned under sustained load at room temperature.

Figure 4.16: Moment versus curvature at midspan for slab strips conditioned under sustained load at room temperature.

Figure 4.17: Midspan moment versus FRP strain at midspan for slab strips conditioned under sustained load at room temperature.

Figure 4.18: Total applied load versus midspan deflection for slab strips subjected to freeze-thaw cycles while under sustained load.

Figure 4.19: Midspan moment versus midspan curvature for slab strips subjected to freeze-thaw cycles while under sustained load.

Figure 4.20: Midspan moment versus midspan FRP strain for slab strips subjected to freeze-thaw cycles while under sustained load.

Figure 4.21: Peak loads for all pull-out tests and average pull-out loads for each specimen type.

Figure 4.22: Peak FRP strains as a fraction of ultimate strain for all pull-out tests and average for each specimen type.

Figure 4.23: Load versus actuator stroke (less FRP elongation) for all room temperature pull-out tests.

Figure 4.24: Load versus actuator stroke (less FRP elongation) for all grout adhesive pull-out tests.
Figure 4.25: Load versus actuator stroke (less FRP elongation) for all epoxy adhesive pull-out tests

Figure 4.26: Typical grout strengthened slab strip failure at the FRP-grout interface (showing crumbled grout at a large flexural crack above the support location and essentially intact grout on either side of the crack)

Figure 4.27: Image of a typical failure of one of the epoxy adhesive strengthened slab strips just below the epoxy/concrete interface, again at the location of a major flexural crack over an internal support

Figure 4.28: Typical failure of a pull-out specimen for the grout adhesive system; the grout remains intact however the strip has slipped at the grout-strip interface

Figure 4.29: Typical failure of a pull-out specimen for the epoxy adhesive system; intact epoxy is visible around the FRP with a thin layer of concrete attached to the epoxy along the bond length. The bend in the FRP indicates that the strip has slipped upwards upon failure, forming the “s” shape due to the additional length between the specimen and the mechanical grips

Figure 5.1: Stress versus strain profile used for the reinforcing steel in the numerical layer model

Figure 5.2: Stress versus strain profiles for the concrete used in the numerical layer model

Figure 5.3: Stress versus strain profile used for the FRP in the numerical layer model

Figure 5.4: Midspan moment versus FRP strain at midspan from room temperature epoxy adhesive strengthened slab strips contrasted with the predictions of the numerical layer model

Figure 5.5: Midspan moment versus midspan curvature data from room temperature epoxy adhesive strengthened slab strips contrasted with the predictions of the numerical layer model

Figure 5.6: Midspan moment versus midspan FRP strain for the epoxy adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model

Figure 5.7: Midspan moment versus midspan curvature for the epoxy adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model

Figure 5.8: Midspan moment versus midspan FRP strain for the grout adhesive strengthened slab strips contrasted with the predictions of the numerical layer model

Figure 5.9: Midspan moment versus midspan curvature for the grout adhesive strengthened slab strips contrasted with the predictions of the numerical layer model
Figure 5.10: Midspan moment versus midspan FRP strain for the grout adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model ................................................................. 151
Figure 5.11: Midspan moment versus midspan curvature for the grout adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model .................................................................................................................................................. 152
Figure 5.12: Peak FRP strains as a fraction of ultimate strain for all pull-out tests and average for each specimen type compared against the Seracino et al. (2007b) model predicted debonding strains ........................................................................................................................................... 152
Figure 5.13: Sample high resolution digital photo taken for the digital image correlation analysis. One pixel patch was placed on the FRP 25 mm above the top of the concrete block, and another patch was placed in the bottom left corner as a zero displacement reference ........................................... 153
Figure 5.14: Applied load versus loaded end slip data from high resolution digital image analysis for the grout pull-out specimens ........................................................................................................... 153
List of Tables

Table 3.1: Properties of Aslan 500, from the manufacturer’s website (www.hughesbros.com) ... 69
Table 3.2: Properties of Kemko 038 Regular IR, taken from the manufacturer’s website ........ 69
Table 3.3: Properties of Target 1118 Grout at 20 °C, taken from the manufacturer’s website ...... 70
Table 3.4: Slab strip testing program ................................................................................... 70
Table 3.5: Direct pull-out testing program .............................................................................. 71
Table 4.1: Results of concrete cylinder tests ......................................................................... 108
Table 4.2: Mechanical properties of Aslan 500 CFRP based on tensile tests reported by Burke (2008) ......................................................................................................................... 108
Table 4.3: Mechanical properties of D5 reinforcing steel based on tension tests reported by Ranger (2007) .......................................................................................................................... 109
Table 4.4: Results from mortar cube compressive strength tests on Target 1118 unsanded silica fume grout adhesive as reported by Burke (2008) ................................................................. 109
Table 4.5: Selected results from the slab strip testing program .............................................. 110
Table 4.6: Maximum FRP strains achieved during pull-out testing ....................................... 111
Table 4.7: Maximum FRP strains achieved during pull-out testing ....................................... 111
Table 5.1: Material properties used in the model ................................................................... 144
Table 5.2: Assumed properties used in the model ................................................................. 144
Table 5.3: NSM groove properties used in the epoxy slab strip model ................................. 144
Table 5.4: NSM groove properties used in the grout slab strip model ................................. 144
Table 5.5: Observed and predicted FRP failure loads and strains for the slab strip testing program ........................................................................................................................................ 145
Table 5.6: Observed and predicted FRP failure strains for the pull-out testing program ....... 146
Chapter 1

Introduction

1.1 General

Concrete is the World’s most common and versatile construction material. One of the main advantages of concrete is its high compressive strength; however, coupled with this is a comparatively low tensile strength. To combat this negative mechanical characteristic, reinforced concrete contains internal steel reinforcement to carry tensile forces. Reinforced concrete (RC) structures are extremely common, and literally form the foundation of all modern urban centres. Because concrete can be formed into almost any shape imaginable, engineers such as Santiago Calatrava (Petroski, 2005) have designed beautiful buildings with graceful curves that are not possible with other building materials. Aside from aesthetic considerations, properly designed concrete structures are also inherently resistant to fire, durable and abrasion resistant, water-tight, and provide excellent thermal and acoustic insulation.

Concrete structures, do not last forever, however, and deterioration due to a variety of interrelated mechanisms and factors is increasingly becoming a liability in modern developed countries. New materials and techniques for construction, repair, rehabilitation, and strengthening of concrete structures are needed.

1.2 Need for Rehabilitation

In some environments over extended periods of time, concrete is vulnerable to various deterioration mechanisms, such as chemical degradation due to sulphates, chlorides, and carbonation, as well as physical degradation due to daily and seasonal freeze-thaw cycles in cold
climates. In the case of freeze-thaw cycling, repeated freezing and thawing can pose potential problems for concrete, such as cracking, scaling, spalling, etc., particularly if proper amounts of air entrainment are not achieved during mixing, casting, and consolidation. Concrete mixes with adequate air entrainment (i.e., 5-8% by volume) are highly durable even in severe freeze-thaw environments (Neville, 1996). In many cases, corrosion of internal steel reinforcement, which results in expansion of the reinforcing steel due to the formation of corrosion products, can magnify freeze-thaw issues by causing additional cracking and spalling of the concrete surrounding the expanding rebar (Broomfield, 1997).

The deterioration of infrastructure in the industrialized world is rapidly becoming a considerable problem (Rizkalla & Hassan, 2002). Out of economic necessity, more stringent inspection and maintenance plans are becoming commonplace in an attempt to maximize and prolong the life spans of existing infrastructure. For example, nearly one in three bridges in the United States is either functionally obsolete (those containing older design features which can no longer accommodate current traffic volumes, as well as modern vehicle sizes or weights) or structurally deficient (those which are composed of deteriorated components are limited to light traffic, or are closed completely due to safety concerns) (ASCE, 2009). The American Society of Civil Engineers has estimated that an investment of $17 billion per year is required to substantially improve current bridge conditions (ASCE, 2009). The aging Rockefeller Road Bridge in Cleveland, Ohio, displayed in Figure 1.1, is a typical example of a decaying North American concrete bridge. Similarly, in Europe, the annual cost associated with the repair of RC bridges due to corrosion of steel reinforcing bars has been estimated at $600 million (USD) (Rizkalla, 2003).
This looming infrastructure crisis is driving research in both industry and academia to develop and optimize methods of structural strengthening and rehabilitation which can prolong or extend the effective lifespan of existing structures.

1.3 Fire-Reinforced Polymers

Fibre reinforced polymers (FRPs) are a distinct class of composite materials; materials which are created by the combination of two or more materials on a macroscopic scale to form a new material which possess superior properties to those of the individual constituents alone (ISIS, 2003). FRPs are composed of high strength fibres embedded in a polymer matrix. The fibres, which are extremely strong, behave in a composite matter with the polymer matrix which binds the entire system together. A graphical representation of the constituent materials in FRP is displayed in Figure 1.2. Typical civil engineering FRPs currently used in North America are usually composed of carbon or glass fibres. Polymers common in structural infrastructure FRPs include epoxies and vinylesters. Compared to steel and other conventional construction materials, FRPs stand out for several reasons including: particularly high strength to weight ratios, outstanding durability and resistance to electrochemical corrosion, ease and speed of installation (particularly in concrete repair and strengthening applications, as described later), electromagnetic neutrality, outstanding fatigue characteristics, and low thermal conductivity (ISIS, 2003).

1.4 Evolution of the Near Surface Mounted Technique

The first application of FRPs in a structural application with reinforced concrete was in Dusseldorf, Germany, in the construction of the Ulenberg-Strasse Bridge. The bridge was post-tensioned with 59 FRP cables, each composed of 19 glass FRP (GFRP) rods (Dhir et al., 1999).
The first FRP infrastructure repair involving concrete was performed in 1991 on the Ibach Bridge, in Switzerland.

For the last 15 years, a great deal of attention has been given to the application of externally bonded FRP sheets as a method of repairing, rehabilitating, or strengthening existing reinforced concrete structures. In these applications, FRP materials are bonded to the exterior of reinforced concrete members, typically using an epoxy adhesive. The externally bonded (EB) method is typically applied in one of two forms: wet lay-up systems and precured systems. Wet lay-up systems, an example of which is shown in Figure 1.3, consist of the application of a dry fibre sheet or fabric, which is impregnated with an epoxy resin on-site and laid up onto the exterior surface of an existing reinforced concrete member. Wet-lay ups can be applied to the tensile face of a beam to improve flexural capacity and/or the sides of a beam to increase shear capacity. Perhaps the most promising application of EB FRP wet lay-up systems is column wrapping, where the FRP material is applied in the circumferential (hoop) direction and therefore confines the concrete and increases the load capacity and axial-flexural deformability of the member. Precured EB FRP systems, on the other hand, use composite FRP shapes (typically plates, rods, bars, and strips) which are manufactured off-site and then bonded to the concrete surface using an epoxy adhesive or putty.

In all EB FRP strengthening systems (except for column wrapping), the bond between the FRP system and the concrete is critical for adequate performance and to fully utilize the high strength of the FRP materials. Thus, for both of the aforementioned approaches to EB FRP strengthening, the surface of the concrete must be sufficiently roughened prior to application of the adhesive to provide a sound surface to promote a strong bond between the FRP and the substrate concrete (this is typically done by light sandblasting). Although EB FRP techniques can
drastically increase the axial, flexural, or shear capacity of a section, debonding failures are a common problem, and numerous attempts and techniques have been made to prevent debonding failures. These brittle failures, which can occur suddenly, with little or no visible warning, often occur at load levels which are considerably lower than the predicted flexural or shear strength of the retrofitted system (Rizkalla et al. 2003). In the case of flexural or shear strengthening applications involving EB FRP systems, some of the available techniques to prevent debonding failures include the use of U-wraps (Rosenboom et al., 2007), FRP anchors (Lam & Teng, 2001), and mechanical fasteners (Lamanna et al., 2004).

In recent years a new method to strengthen reinforced concrete structures has emerged involving application of FRP in the ‘near surface’ of a reinforced concrete member (i.e., set into slots or grooves cut into the concrete cover). This technique has become known as the near surface mounted (NSM) technique (Blaschko & Zilch, 1999). The method entails cutting a groove or slot into the concrete cover of a member, and then placing a precured FRP bar, rod, strip, or plate, along with an epoxy adhesive, into the groove as shown in Figure 1.4. Most NSM FRP installations are performed on reinforced concrete structures; however, the process is also suitable for some timber (De Lorenzis & Nanni, 2002) and masonry (Fam et al., 2003) strengthening projects. Advantages of the NSM technique over EB systems include minimal surface preparation and installation time, the ability to anchor the FRP into an adjacent member (thus improving bond performance), superior protection from mechanical and environmental damage, and superior bond properties (i.e., reduced risk of undesirable debonding failures, thus utilizing a greater proportion of the FRP’s strength). The NSM technique is especially appealing for negative moment strengthening of concrete members, where NSM FRP has less impact on the presence of floor finishes, overlays, or wearing surfaces than traditional EB FRP systems.
Although NSM FRP has only recently seen widespread attention for rehabilitation using FRPs, the method has been used for over a half century. In 1948, a reinforced concrete bridge being constructed in Sweden experienced excessive settlement of the negative moment reinforcement during casting, such that the negative moment capacity of the section needed to be increased. To achieve this, grooves were cut in the cover of the concrete and then filled with cement mortar and steel rebar. Research done prior to the bridge installation revealed that the behaviour of NSM strengthened sections was virtually identical to the RC sections as had originally been designed (Asplund, 1949).

1.5 Research Significance

Although a reasonably large number of laboratory studies, field applications, and experimental field projects have employed the NSM FRP strengthening technique, to the knowledge of the author none of these have been performed in a climate where the potential damage from cold environments and freeze-thaw cycling could be issues. Strengthening applications using NSM FRP in cold climates cannot be performed with confidence until studies have examined the behaviour of the NSM FRP systems, both in terms of bond performance and in terms of overall member performance, after exposure to harsh environmental conditions. The research presented herein will add to the existing knowledge base of NSM FRP bond behaviour, and also examines some of the as yet unexamined cold climate durability issues which still surround the application.

1.6 Research Objectives

The objective of the research presented herein is to gain a better understanding of the behaviour of NSM FRP strengthening systems for reinforced concrete members subjected to repeated freeze-thaw cycles, as would be experienced in an application with an exterior exposure
in a cold, temperate, or alpine climate. The primary objectives treated in the current document are:

- to experimentally investigate the performance of flexural NSM FRP strengthening systems for reinforced concrete slab strips after repeated freeze-thaw cycling while under sustained service load, as might realistically be experienced in bridge or parking garage slab strengthening applications in Canada;
- to examine the load transfer and failure behaviour of the bond between NSM FRP and concrete bonded with either epoxy or cementitious grout and investigate the relative performance of both epoxy and cementitious adhesives for use in flexural and/or shear NSM FRP strengthening applications for reinforced concrete members at room temperature and after exposure to repeated freeze-thaw cycles, with a view to reducing installation costs by using lower cost adhesive systems; and
- to study the behaviour of NSM FRP bond in flexural strengthening applications (as tested using notched slab tests) versus a shear strengthening application (as tested using a modified pull-out test).

The secondary objectives are:

- to compare the observed bond performance in the current study against the predictions of available models to predict the bond performance of NSM FRP strengthening systems for concrete; and
- to attempt to utilize a novel digital image correlation technique to analyze bond performance of the specific NSM strengthening system used in the current study, with either epoxy or cementitious adhesive systems, and to correlate these data with those obtained using conventional instrumentation.
1.7 Scope

The research program presented herein involved the fabrication and testing of 21 medium scale notched reinforced concrete slab strip specimens and 16 modified NSM bond pull-out specimens. For both cases, half of the specimens used an epoxy adhesive to bond the NSM system, while the other half used cementitious grout adhesive. The slab strips measured 1524 mm in length with a cross section width and height of 254 mm and 102 mm respectively. The slabs were all strengthened with a commercially available NSM carbon FRP strip system and tested to failure in flexure after exposure to one of the following conditions: room temperature under zero applied load (controls), room temperature under sustained service load, 300 freeze-thaw cycles without load, or 300 freeze-thaw cycles under sustained service load. The modified bond pull-out specimens were tested after exposure to either room temperature conditions or 150 freeze-thaw cycles, in both cases without sustained load.

Tests were performed by other students within the author’s research group to determine the mechanical properties of the constituent materials, including tensile tests on the steel reinforcing bar and NSM carbon FRP tape at room temperature. Compressive tests were also performed on concrete cylinders after room temperature exposure and after 300 freeze-thaw cycles.

Photogrammetry (high resolution digital image tracking software) was used in an attempt to observe the loaded end bond-slip of the FRP during testing of the modified pull-out specimens.

The results of the current study were compared against experimental data and bond models available in the literature, and the consequences of the observed behaviour for the performance of NSM FRP-strengthened concrete members in cold climates are discussed.
1.8 Outline of Thesis

This thesis is formatted according to the “Traditional” format described in the Queen’s University School of Graduate Studies General Forms of Theses. A summary of each chapter is provided below:

**Chapter 2 (Literature Review)** presents background information on the use of NSM FRPs. The results from previous tests involving both NSM rods and strips are discussed. Additionally, several field applications are presented. The behaviour of the materials in a reinforced concrete/FRP system are presented under conditions of freeze-thaw cycling and sustained load. Available bond models are examined along with the existing NSM FRP strengthening guidelines.

**Chapter 3 (Experimental Procedure)** describes the test program performed to study the NSM FRP systems after various exposure conditions formed from combinations of ambient conditions, freeze-thaw cycling, and sustained load. The chapter is broken up into two sections. First, the test procedure for the slab strip testing program is discussed. Second, the test procedure of the modified pull-out testing program is summarized. In both cases, the design, fabrication, and NSM FRP strengthening systems are detailed. The testing setup, testing procedure, and instrumentation used during testing are also reported. Finally, ancillary tests to characterize the materials used are summarized.

**Chapter 4 (Results and Discussion)** presents the data obtained during the testing program. The results are summarized in tables and graphs comparing the performance of the NSM FRP systems with exposure to a period of sustained load and/or freeze-thaw cycling to those without.
Chapter 5 (Analysis) presents the comparison of NSM FRP bond models available in the literature with the experimental data collected during testing. Finally, an equilibrium based numerical layer model created in Microsoft Excel is used to compare the observed and theoretical moment versus curvature responses of the NSM FRP strengthened slab strips.

Chapter 6 (Conclusions and Recommendations) summarizes the important findings of the research program presented in this thesis and makes recommendations for further research which would benefit the knowledge base surrounding NSM FRP.
Figure 1.1: The aging Rockefeller Road Bridge in Cleveland, Ohio, is one of many decaying RC bridges (ASCE, 2005)

Figure 1.2: Constituent materials of FRP (ISIS Canada, 2003)
Figure 1.3: Shear strengthening with Externally Bonded (EB) wet lay-up FRP sheets (Rizkalla et al., 2003)

Figure 1.4: Applying Near Surface Mounted (NSM) FRP rods (Rizkalla et al., 2003)
Chapter 2

Literature Review

2.1 Motivation

North American infrastructure is deteriorating rapidly. A decade ago, the Secretary of Transportation reported to the Congress of the USA that nearly 40% of inventoried highway bridges located on public roads were built before 1940 (El-Hacha, 2000). The latest American Society of Civil Engineers (ASCE) Report Card, published in 2009, states that the United States requires $2.2 trillion dollars in infrastructure upgrades. Of the America’s 600,905 bridges, 26.9% (161,892 bridges) were deemed as structurally deficient or functionally obsolete (ASCE, 2009).

The deterioration of infrastructure is a similar concern in Canada and in much of the rest of the industrialized world. For example, the collapse of a highway bridge near Laval, Quebec in 2006 served to highlight the need for better design, construction, inspection, and repair of Canadian infrastructure systems. The report on the investigation of this specific collapse highlighted that 49% of all bridges in Quebec are considered deficient, as compared with 32% in Ontario. Among the 17 recommendations made in the report, it was stated that $500 million per year for at least ten years needs be spent on bridge and overpass repair and construction in Quebec alone. The development of a national bridge rehabilitation program was also suggested (Commission of Inquiry into the Collapse of a Portion of the de la Concorde Overpass, 2007). Although specific and quantifiable data for the rest of Canada are not readily available, Canada’s estimated 200,000 deficient bridges (circa 2000) would cost an estimated $44 billion to replace (El-Hacha, 2000). Furthermore, the rehabilitation of Canadian concrete parking structures alone
has been estimated (again circa 2000) to cost between $5 billion (El-Hacha, 2000) and $6 billion (Benmokrane and Wang, 2001).

2.2 NSM Background

Fibre reinforced polymers have found a wide array of applications in infrastructure strengthening, including repair, retrofit, reinforcing, and strengthening of masonry, timber, steel, and concrete structures. In repair and strengthening applications the materials are typically bonded using an epoxy resin to the external surface of a structural member. Depending on the specific location and orientation of the bonded FRPs, this technique can be used to increase flexural strength, shear strength, or axial strength of concrete members. However, a more recent development in the use of FRPs in repair and strengthening of concrete, masonry, and some timber structures involves FRP application by adhesive bonding within a groove in the concrete cover; i.e., in the near-surface of the structural member.

Near surface mounted (NSM) FRP reinforcement has been proven to be an efficient and effective strengthening technique for deteriorated or deficient reinforced concrete structures. Both the flexural and shear strength of reinforced concrete flexural members can be increased by inserting FRP rods or strips, typically along with an epoxy adhesive, into grooves cut in their concrete cover. Near surface mounted FRP reinforcement has several key advantages over externally-bonded FRP repair systems, including:

- the possibility of anchoring the FRP into adjacent members;
- minimal surface preparation;
- reduced installation time;
- no requirement for protective coatings;
- resistance to vandalism or other inadvertent damage mechanisms; and
particular applicability to regions of negative bending where external reinforcement is undesirable (De Lorenzis and Teng, 2007).

Also, compared to externally bonded FRPs, NSM systems provide greater ultimate load capacity and greater deformation capacity at failure, largely as a consequence of their superior bond performance (Barros et al., 2007).

Research in the area of NSM FRP strengthening systems for reinforced concrete structures has advanced to the point where specific design guidelines for these systems are available, most notably from Committee 440 of the American Concrete Institute (ACI, 2008). Academic interest in FRPs has been paralleled in industry with several large scale field applications including strengthening of both bridges and silos (Parretti & Nanni, 2004, Nanni, 2000).

Although numerous applications of NSM FRP systems have been realized, many uncertainties remain with respect to their performance and durability in cold climates and under sustained load. For instance, before NSM FRP reinforcement can be applied with confidence in conditions exposed to harsh environments, such as those experienced in exterior applications in Canada and other cold or temperate regions, several important issues surrounding their long-term durability must be investigated. Specifically, research is needed to probe the long-term freeze-thaw durability of NSM systems in cold climates, which remains, to the knowledge of the author, entirely un-investigated.

2.2.1 NSM Rods

Although the focus of the research program presented in this thesis is to investigate the performance of NSM FRP strips installed in thin, deep grooves cut into the concrete cover, it is worth noting that several researchers have examined the performance of circular or square NSM
FRP rods in both shear and flexural strengthening applications. Indeed, much of the early work on both NSM steel and NSM FRP reinforcement was performed using round bars installed in square (or rectangular) surface grooves (De Lorenzis & Nanni, 2001, De Lorenzis et al., 2002, De Lorenzis et al., 2004, Hassan & Rizkalla, 2004).

In the case of shear strengthening with NSM FRPs, where the FRPs are bonded into the side faces of a concrete member, early work on NSM FRP bars indicated that the performance of the strengthened members depends on, amongst other factors, the spacing of the rods and the orientation of the rods with respect to the beam. For instance, reinforced concrete beams without internal steel shear reinforcement have had their shear capacity increased by up to 41% after strengthening using vertical NSM carbon FRP rods bonded with an epoxy adhesive (De Lorenzis and Nanni, 2001). Rotating the orientation of the NSM FRP from vertical to a 45 degree angle increased the capacity of the sections by 97% compared to the control specimens. Vertical CFRP rods anchored into the flanges of T-beams have proven most effective, increasing the shear capacity by 106% compared to the control specimens. It would be natural to assume that NSM FRP rods anchored into the flange at an orientation of 45 degrees would produce even more efficient use of the material, but this scenario has apparently yet to be evaluated. De Lorenzis and Nanni (2001a) also noted that NSM FRP rods can be used effectively to increase shear capacity by up to 35% when internal steel stirrups are present.

Early work using NSM FRP bars has also shown that both the flexural stiffness and flexural strength of reinforced concrete members can be increased using NSM FRP systems. Ultimate load capacity increases of up to 41% have been achieved for NSM carbon FRP bar strengthened T-beams using an epoxy adhesive (Hassan and Rizkalla, 2004). The modes of failure for all of the strengthened beams in this study were due to splitting of the concrete surface
at the concrete-epoxy interface. The ultimate stress in the FRP bars increased as the NSM embedment length increased. However, the relative increase in ultimate stress in the FRP diminished with increasing embedment length. For example, increasing the embedment length produced appreciable increases in tensile stress of the FRP up to a length of 800 mm. When the embedment length was increased to 1200 mm, the tensile stress in the FRP at failure was only 7.5% greater than that produced with an 800 mm embedment length. This suggests that there is an embedment length beyond which no increase in strength will be observed, much the same as has been observed for externally bonded FRP plates and sheets (Bisby, 1997).

The modified pull-out test used in the research presented in this thesis (discussed in detail in Section 3.3 and shown in Figure 2.1) was originally developed for bond tests on round NSM FRP bars by De Lorenzis et al. (2002). The C-shaped concrete block has a groove for embedment of an NSM FRP rod which, after installation, is located at the centre of the cross-section. The applied tensile load in the FRP is transferred to four threaded steel rods located at each corner of the cross-section, thus preventing artificial compressive confinement of the bond region during testing. This testing setup was designed to allow visual access to the testing zone during loading while also allowing for the measurement of both loaded-end and free-end slip. In addition, the setup eliminated the prior need for two bonded lengths of FRP bars for reaction, thus minimizing the preparation time and potential problems arising from eccentric loading. Research by De Lorenzis et al. (2004) revealed that the optimal groove size to rod diameter ratio was approximately two for deformed circular FRP bars. For groove size to rod diameter ratios less than two, failure tended to be the result of tensile splitting of the adhesive, but when the ratio was greater than two, cohesive shear failure in the concrete was more common (refer to Figure 2.2). Furthermore, as expected, epoxy adhesive displayed superior mechanical performance as
compared with cement mortar as the adhesive groove filler. The maximum FRP stress in the epoxy bonded specimens was 66% of the ultimate FRP tensile stress, whereas the cement mortar-filled grooves used much less of the FRP’s tensile strength, reaching an ultimate stress of only 27% of the tensile capacity of the rod.

2.2.2 NSM Strips

Of the various cross-sectional shapes of NSM FRP reinforcement which have been investigated in the literature, including round bars, square rods, rectangular rods, and narrow strips, it appears the latter are least prone to debonding from the concrete substrate (El-Hacha and Rizkalla, 2004). The superior performance of NSM strips (sometimes called tapes) is twofold. First, thin strips maximize the ratio of surface area to cross-sectional area, which minimizes the bond shear stresses for a given tensile force in the FRP. Second, as shown in Figure 2.3, the normal stresses accompanying the tangential stresses in the adhesive act against either side of the strip into the concrete substrate, thus effectively confining the strip and improving bond performance. In the case of round FRP bars, the normal stresses act outward in all directions, eventually causing the epoxy cover to split when the tensile strength of the adhesive is reached (Blaschko, 2003). Local bond strengths in NSM strips as high as 18 MPa have been observed (De Sena Cruz and Barros, 2004), which is greater than typical bond strengths of externally bonded systems or NSM systems using round rods (Teng et al., 2006).

In flexural applications, NSM FRP strips have been used to increase the ultimate capacity of a beam by as much as 106% (Hassan and Rizkalla, 2003; Barros and Fortes, 2004; Teng et al., 2006). In some cases, the FRP strips have been used to full composite action where failure of the section was the result of FRP strip rupture in tension (Hassan and Rizkalla, 2003). In most cases,
however, debonding by one of a number of interrelated mechanisms occurs prior to generating the ultimate tensile stress in the NSM FRP.

From a shear strengthening perspective, NSM strip systems are also particularly attractive in terms of attaining the greatest effectiveness of the FRP. Beams strengthened for shear using NSM strips have shown similar ultimate capacities to beams with equivalent reinforcement ratios of internal steel stirrups. Furthermore, again using equivalent reinforcement ratios, NSM strip systems have presented an average increase in shear capacity of 83%, compared to an average increase of only 54% for EB FRP systems, over control specimens without shear reinforcement (Barros et al., 2007).

2.2.3 Field Applications

One of the earliest field applications of NSM FRP was for strengthening a reinforced concrete bridge near Rolla, Missouri (Nanni, 2000). Due to realignment of an existing highway, the bridge was set to be decommissioned in December 1998. After a field inspection, it was realized that the 1930s bridge was still in good condition and provided an excellent opportunity for full-scale field tests. The bridge consisted of three simply supported spans. One span was strengthened with NSM CFRP rods (as shown in Figure 2.4), and another span was strengthened with externally bonded CFRP sheets. The remaining section was left as a control span without strengthening. The entire retrofit of the bridge took three workers seven days to complete, without interrupting traffic. Testing of the bridge revealed the NSM strengthened deck provided a 27% increase in moment capacity, which was greater than the 17% increase achieved by the CFRP sheet strengthening system.

In a similar field research partnership, reinforced concrete silos (shown in Figure 2.5) were strengthened with NSM carbon FRP rods to meet updated building code requirements.
Traditional techniques for silo strengthening include adding post-tensioned external steel cables, or adding a reinforced concrete wall inside (an interior sleeve) or outside (an exterior sleeve), typically applied using shotcrete. Near surface mounted FRP was chosen as the method of strengthening the silo for two main reasons. First, the quick installation on the exterior surface of the silo minimized service interruption of the structure. Adding an additional RC wall on the exterior of the structure would have required the entire surface be sandblasted to provide a suitable surface for shotcrete bond. Second, the storage capacity of the silo was not reduced, as would have occurred if another internal RC sleeve had been formed inside the existing structure. The project, which was presented in a paper to the International Concrete Repair Institute (ICRI), received the Award of Excellence in the Industry Category (Prota et al., 2003).

### 2.2.4 Bond Failure Modes

Near surface mounted FRP strengthened concrete members may display one of a variety of possible failure modes. Those highlighted in the literature to date are:

- Concrete crushing in the compression zone after yielding of the longitudinal steel reinforcement – is the preferred method of failure in flexural FRP design because it provides the greatest warning of failure (ACI, 2008). This type of failure is not specifically examined in the NSM FRP literature because it does not provide information specific to the failure caused by the FRP itself, and relies solely on the properties of the cross-section, assuming perfect bond between the FRP and the concrete. In practical design situations, the ultimate strain in the FRP is typically limited to a value of 70% of the manufactures’ guaranteed ultimate tensile strain (ACI, 2008) to increase the probability of failure in the concrete before bond failure or tensile rupture of the FRP.
• Splitting of the adhesive cover – is fairly common in specimens strengthened using NSM deformed FRP rods. As for the bond between steel reinforcement and concrete, the deformed shape of the NSM FRP rod transfers much of the load to the surrounding material by mechanical interlock. This load transfer results in stresses in both the longitudinal and radial directions with respect to the rod. When the stresses in the radial direction exceed the tensile strength of the adhesive which contains it the adhesive splits longitudinally in the direction perpendicular to the length of the rod (De Lorenzis and Nanni, 2001; Hassan and Rizkalla, 2004; De Lorenzis et al., 2004).

• Splitting of the concrete cover along the internal longitudinal steel reinforcement (known as cover delamination) – has been observed in both shear and flexural strengthening applications. In a shear strengthening scenario, splitting of the concrete cover has occurred in studies when the positioning of the NSM strips was altered from vertical to a 45° inclination, providing a longer bond length. In addition to the incline, the latter series of strips were also anchored into the flange. The additional bond length prevented bond failure, and forced the failure to occur along the surface of the horizontal reinforcing steel (DeLorenzis and Nanni, 2001). In a flexural strengthening application, loss of the concrete cover has been observed, particularly in cases where the beams were strengthened beyond what would be expected in engineering practice. For example, two beams failed by cover separation at loads of 91% and 96% greater than their unstrengthened partner beams (Barros and Fortes, 2005). Current strengthening limits (ACI, 2008) would prevent such high levels of strengthening in all practical situations. The first signs of this type of failure are longitudinal cracks in the concrete at the location of highest FRP stress, running parallel to the NSM FRP near the level of the internal
longitudinal steel reinforcement. Gradually, the cracking progresses towards the position of lowest FRP stress, and eventually rips out a piece of the concrete cover. This type of failure is sudden and clearly undesirable (De Lorenzis and Nanni, 2001).

- Adhesive pull-out – has been experienced in pull-out tests which used cast-in-place grooves to accept the NSM FRP reinforcement in purely research applications, rather than cut grooves using a diamond blade as would occur in real field applications. In this case, the smooth interior surface of the grooves prevented the sound surface bond that would have more likely been achieved if the grooves were cut using a diamond blade (De Lorenzis et al., 2002). For real strengthening applications this would obviously never occur, as a groove would have to be cut into the concrete cover to apply the NSM FRP in all cases. Indeed, the same test setup was repeated two years later with diamond blade grooves cut into the specimens rather than cast-in-place grooves, and the pull-out failures were prevented (De Lorenzis et al., 2004).

- Tensile rupture of CFRP strips – although rare, has been observed in a few research programs. In a test setup which evaluated the effects of varying bond length on beams strengthened in flexure, Hassan and Rizkalla (2003) observed rupture in all tests with NSM groove embedment lengths greater than 850 mm. This was the first time that NSM FRP rupture in a strengthening application was observed in the laboratory. From a design perspective this type of failure is least desirable because it is sudden and violent. However, from a research perspective, this was the first time NSM FRP strips were used to their full potential, and it therefore represents the most economical use of the FRP material. More recently, a beam strengthened in shear using vertical CFRP strips failed
by FRP rupture after a large shear crack propagated through the middle of one of the NSM strips (Barros et al., 2006).

In general, NSM FRP flexurally strengthened beams seem to fail by concrete cover separation. As the width of the section increases (and therefore the failure plane if failure is by cover separation), such as in slabs, the failure shifts to debonding by rupture of the concrete immediately adjacent to the adhesive. Shear strengthening NSM installations tend to fail by debonding.

2.2.5 Bond Models

The evolution of NSM FRP research has seen the development of several models to predict the response of an NSM FRP bonded length (for use in both flexural and shear strengthening applications). Early models were calibrated against very specific test setups used by each respective research group, and were thus relatively impractical for wider use by the engineering design community. Recently, a more widely versatile model for the bond strength of adhesively bonded plates to concrete has been developed (Seracino et al., 2007a). The model predicts the intermediate crack (IC) induced debonding strain (or stress) of an NSM plate.

Intermediate crack induced debonding (shown schematically in Figure 2.12) occurs when a shear or flexural crack in the reinforced concrete section reaches the NSM strip or externally bonded FRP plate, eventually causing the propagation of bond failure away from the crack tip and toward the plate’s free ends (Seracino et al., 2007a). Intermediate crack-induced debonding is recognized as the most important of the major forms of debonding for reinforced concrete structures strengthened with adhesively bonded plates (Oehlers and Seracino, 2004). A model proposed by Seracino et al. (2007a), was derived for the cases of both externally bonded and NSM FRP plates and strips with bond lengths greater than or equal to the critical bond length.
The model is a function of the material and geometric properties of the section being analyzed, and can be applied to any bonded plating technique and plate material. Parameters used in the Seracino et al. (2007a) bond model are shown graphically in Figure 2.6. The aspect ratio, \( \varphi_f \), highlights one of the benefits of NSM FRP over externally bonded FRP. The aspect ratio is in essence a measure of confinement of the failure plane by the concrete which surrounds it. In cases with small aspect ratios, such as with the EB technique, the surrounding concrete provides very little confinement. However, in cases with large aspect ratios, such as with the NSM strip technique, the surrounding concrete cover provides excellent confinement and generates friction and aggregate interlock across the eventual failure plane, which is typically in the concrete. The model was derived using equilibrium and compatibility of a plate-to-concrete joint with statistical analysis against push-pull data available in open literature. Under the conditions listed above, the following expression can be used to predict the load level at which IC debonding occurs (Seracino et al., 2007a):

\[
P_{IC} = \alpha_p 0.85 \varphi_f^{0.25} f_c^{0.33} L_{per}(EA)_p < \begin{cases} f_{ult} A_p & \text{for FRP plates} \\ f_{p,c} A_p & \text{for metallic plates} \end{cases}
\]  

[Eq 2.1]

where:

- \( P_{IC} \) = maximum load in the strip before debonding (N)
- \( \alpha_p = \) \begin{cases} 1.0 & \text{for mean} \\ 0.85 & \text{for lower 95% confidence limit} \end{cases} 
- \( \varphi_f = \frac{d_f}{b_f} \) = aspect ratio of the interface plane
- \( t_p = t_d = 1 \text{ mm} \) (depth from the adhesive to the failure plane in the concrete)
This model does not explicitly take into account the properties of the adhesive. This is because it is assumed the adhesive is sufficiently strong and stiff such that the debonding failure plane will propagate through the concrete rather than the adhesive. It should be noted that this has only been verified for polymer adhesives, and there is some doubt that it would hold for cementitious adhesives or grouts, such as the product used for some of the tests presented later in this thesis.

\[ d_f = \text{depth of groove} + t_d \]

\[ = \text{depth of the failure plane perpendicular to the concrete surface (mm)} \]

\[ b_f = \text{width of groove} + t_b \]

\[ = \text{width of the failure plane parallel to the concrete surface (mm)} \]

\[ L_{per} = 2d_f + b_f \]

\[ = \text{length of the failure perimeter (mm)} \]

\[ f'_c = \text{concrete compressive strength (MPa)} \]

\[ E_p = \text{elastic modulus of plate (MPa)} \]

\[ A_p = \text{area of plate (mm}^2) \]

\[ f_{ult} = \text{ultimate strength of FRP plate (MPa)} \]

\[ f_y = \text{yield stress of steel plate (MPa)} \]
Another model, also developed by Seracino et al. (2007b) was designed to also include cases of bond length less than the critical bond length and to represent a lower bound bond strength for such cases. Similar to Seracino et al. (2007a), the model determines the IC debonding load. The model, given below in Equation 2.2, was developed based on a statistical analysis from 36 push-pull tests using only NSM CFRP strips (ie EB plates were not used in the derivation of the model). The equation suggests that the concrete strength and NSM FRP dimensions play dominant roles in governing the IC debonding load.

\[ P_{IC} = \alpha \beta \sqrt{f_c} \sigma_p^{1.35} \beta_p^{0.21} \leq f_{ult} \beta_p d_p \]  

[Eq 2.2]

where:

\[ \alpha = \begin{cases} 
0.19 & \text{for mean value} \\
0.16 & \text{for characteristic value} 
\end{cases} \]

\[ \beta = \begin{cases} 
1.0 & \text{for } L \geq 200 \text{ mm} \\
\frac{\beta}{200} & \text{for } L < 200 \text{ mm} 
\end{cases} \]

\[ d_p = \text{dimension of the FRP plate parallel to the concrete surface (mm)} \]

\[ \beta_p = \text{dimension of the FRP plate perpendicular to the concrete surface (mm)} \]

An analytical NSM bond model based on combined shear and bending was proposed by Hassan & Rizkalla (2003). The model assumed that high shear stresses at the end of the NSM reinforcement caused debonding, rather than failure which was initiated at intermediate flexural or shear-flexure cracks. The model produces a shear stress for a given scenario which must then be compared against a maximum allowable shear stress. The limitation of this model, however,
is that the maximum allowable shear stress for NSM systems has yet to be accurately determined, either computationally or experimentally. Bond stresses observed in tests have ranged from as little as 3.5 MPa to as large as 20.7 MPa for NSM FRP systems (ACI, 2008). Furthermore, the ultimate shear stress for each FRP configuration and surface texture would be different. In addition to the unknown bond stress values, the model is of little practical value because it was only calibrated for the specific setup examined in these tests. The three forms of the closed-form analytical solution proposed to predict the interfacial shear stresses are summarized below:

For a simply supported condition and midspan loading:

$$\tau = \frac{t_f}{2} \left( \frac{nP l_o y_{eff}}{2l_{eff}} \omega \epsilon - \omega \chi + \frac{nP y_{eff}}{2l_{eff}} \right)$$

where:

$$\tau = \text{interfacial shear stress}$$

$$t_f = \text{thickness of the FRP strip}$$

$$n = \frac{E_f}{E_c} = \text{modular ratio}$$

$$P = \text{applied concentrated load}$$

$$l_o = \text{unbonded length of FRP strip}$$

$$y_{eff} = \text{distance from the FRP strip to the neutral axis}$$

$$l_{eff} = \text{effective moment of inertia of the transformed section}$$
\[ \omega = \sqrt{\frac{2G_a}{t_a t_f E_f}} \]

\[ t_a = \text{thickness of the adhesive layer} \]

\[ G_a = \text{shear modulus of the adhesive} \]

\[ \chi = \text{longitudinal coordinate starting from cutoff point of FRP strip} \]

For a simply-supported condition for the member and uniformly distributed loading:

\[ \tau = \frac{t_f}{2} \left( a\chi + b + c\omega e^{-\omega \chi} \right) \]

where:

\[ a = -\frac{qny_{eff}}{2l_{eff}} \]

\[ b = \frac{qny_{eff}}{2l_{eff}} \left( L' - 2l_o \right) \]

\[ c = -\frac{qny_{eff}}{\omega^2 l_{eff}} + \frac{qny_{eff} l_o}{2l_{eff}} \left( L' - l_o \right) \]

\[ q = \text{applied uniform load} \]

\[ L' = \text{total span of simply supported beam} \]

or for a simply-supported condition and two equally spaced concentrated loads:

\[ \tau = \frac{t_f}{2} \left( \frac{nPy_{eff}}{l_{eff}} + \frac{nPy_{eff} l_o}{l_{eff}} \omega e^{-\omega \chi} \right) \]
The failure limit for the previous three scenarios is based on the properties of the concrete as:

\[ \tau_{\text{max}} = \frac{f'_c f_{ct}}{f'_c + f_{ct}} \]

where:

- \( f'_c \) = compressive strength of concrete
- \( f_{ct} \) = tensile strength of concrete

The generic intermediate crack induced debonding model by Seracino et al. (2007a) is used in Chapter 5 of this thesis to compare against the experimental data which were collected during testing of the slab strips. This specific model was chosen for comparison because of its generic nature, which lends itself naturally to a variety of test setups. The IC debonding model developed specifically for NSM strips (Seracino et al., 2007b) was used to compare against the pull-out bond tests because this model was developed using a push-pull test of a similar nature. Furthermore, this model also accounts for bond lengths which are less than the critical bond length (as is likely the case for the 150 mm bond length of the pull-out specimens of the current thesis).

2.3 Freeze-Thaw Effects on Structural Materials

In areas where the environmental temperature fluctuates above and below the freezing point of water, freeze-thaw damage of structural elements can be an issue for structures in service. It is not simply the freezing of water, however, that causes concerns. When two or more materials are combined into a single structural element, issues can arise with temperature changes
because of differential thermal expansion between the two materials. The freeze-thaw effects on concrete, FRP, and adhesive materials are discussed on this section.

2.3.1 Concrete

Aside from chloride induced corrosion, freeze-thaw cycling is the most common type of concrete deterioration in cold climates. The cracking, crumbling and scaling associated with freeze-thaw damaged concrete is largely the result of water expansion within the concrete upon freezing. In areas where the concrete voids are filled with water, pressure develops within the voids as the water freezes and expands by between eight and nine percent. If sufficient space is not available within the concrete for the expanding ice, it generates an internal pressure within the cracks or voids within concrete that exceed its tensile strength, and the concrete will crack. Repeated freezing and thawing under these conditions can cause severe deterioration of concrete.

To help minimize the damage to concrete from freeze-thaw cycling, air can be intentionally entrained within the concrete matrix during fabrication. It has been widely known since the 1940s that the freeze-thaw durability of concrete can be greatly improved by air entrainment (Hobbs, 2001). Tiny, spherical air voids provide additional space for pore water to expand, therefore relieving the internal pressures that arise during ice formation and preventing cracking. An example of air-entrained and non-air-entrained concrete can be seen in Figure 2.7, which shows the entrained air voids required for satisfactory freeze-thaw resistance. Air is entrained in the form of microscopic air bubbles by the addition of an air-entraining admixture during the manufacturing of the concrete. The amount of entrained air is typically between five and eight percent by volume of the concrete (Portland Cement Association, 2010). The concrete used in this thesis was air-entrained within this range, thus the freeze-thaw durability of the concrete itself was not expected to influence the results.
2.3.2 FRPs

Exposure to cold temperatures or freeze-thaw cycles can cause problems for FRPs on two levels. First, the coefficients of thermal expansion (CTEs) of the FRPs’ fibres are typically different from those of their polymer matrices. The differing rates of thermal expansion of the fibres and the matrix on heating or cooling therefore induce internal stresses at the fibre-matrix interfaces. In addition, FRPs’ bond to concrete can be affected by the different longitudinal and lateral CTEs of concrete and FRP. Second, polymer embrittlement can cause the strength and stiffness of the polymer matrix to increase, so that bond failure modes could become more brittle. Polymer embrittlement may also reduce the ability of the matrix to transfer stresses to the individual fibres, or between the composite and the substrate concrete (Green et al., 2006).

Micelli (2004) performed a study to improve the knowledge base of civil engineering related cold-regions durability data for FRPs by investigating the effects of accelerated aging on FRP rods used in civil engineering applications. A set of five types of carbon and glass FRP bars (shown in Figure 2.8) were exposed to various conditions, including freeze-thaw cycling, high relative humidity, high temperature, and ultraviolet radiation (Micelli, 2004). A total of 50 specimens were subjected to environmental cycles: 20 tensile specimens and 30 ASTM D4475 specimens. The ASTM D445 test setup is a small bending test and is shown in Figure 2.9. The testing regime consisted of cycling from -18°C to 4°C in accordance with the ASTM C666-92 freeze-thaw cycling standard. Following the freeze-thaw cycles, a series of high temperature cycles, varying from 16°C to 49°C, were alternated with constant temperature periods with varying humidity levels. During the high temperature and high relative humidity cycles, the specimens were also exposed to UV radiation. The environmental cycle diagrams shown (Figure 2.10) were performed four times for each specimen. In total, each specimen was subjected to 200
freeze-thaw cycles (50 times four) by the end of the exposures. In addition to freeze-thaw, 480 humidity cycles (120 times four) and 600 high temperature cycles (150 times four) were also performed.

Tensile tests indicated that the longitudinal mechanical properties of the FRPs were not damaged after the environmental cycles. The glass FRPs indicated a slight reduction in average strength after cycling, but the variation in the data makes this inconclusive. The tensile properties of the carbon FRPs were apparently not affected by the environmental cycling.

The transverse properties, which were tested by the ASTM D4475 tests, are most influenced by the polymer resin’s properties. The potential of damage to the resin after environmental cycling was tested by determining the apparent horizontal shear strength of the specimens. In all scenarios, however, the residual strengths were the same as the control strengths, indicating that this particular environmental cycling regime did not significantly alter the FRPs’ horizontal shear strength.

2.4 Freeze-Thaw Effects on FRP Strengthened Concrete Members

Although the freeze-thaw durability of NSM strengthened concrete has yet to be studied, several studies have examined the performance of externally bonded FRP strengthened reinforced concrete members after exposure to environmental (freeze-thaw) cycling. Results from tests on pull-off specimens have suggested that freeze-thaw cycling does not reduce the ultimate load capacity of the joint between concrete and CFRP plates (Green et al., 2000). It was noted, however, that the failure mode shifted from failure in the concrete in specimens without environmental cycling, to failure in the adhesive and even into the CFRP plate in some cases, suggesting the adhesive is slightly affected by freeze-thaw action. Similar testing by Colombi et
al. (2010) concluded that the bond of CFRP plates to concrete using Sikadur 30, specifically, is not weakened by 200 freeze-thaw cycles.

Small scale beams strengthened in flexure with CFRP or GFRP have displayed no discernable damage to member performance after exposure to freeze-thaw cycling (Green et al., 2000; Green et al., 2003). Furthermore, both ISIS Canada (ISIS, 2001) and ACI 440 (ACI, 2008) guidelines for flexural strengthening of reinforced concrete beams with externally bonded FRP plates or sheets conservatively predict flexural capacity even after freeze-thaw cycling (Green et al., 2003). Freeze-thaw cycling in combination with sustained load also appears to produce no discernable negative effect on ultimate load (Oldershaw, 2008). FRPs used in column wrapping (confinement) applications have also displayed excellent resistance to combined freeze-thaw and sustained load (Kong et al., 2005). Although externally bonded FRP has shown excellent resistance to the potential damage caused by freeze-thaw cycling, near surface mounted FRP has yet to be examined in this context. Externally bonded systems use a thin layer of adhesive in contrast to NSM systems which have a concentrated volume of adhesive in a groove in the concrete cover. The larger the volume of adhesive used, the greater the problem of differential thermal expansion, and the greater potential for bond degradation after freeze-thaw cycling. Research into the specific issue of freeze-thaw durability for NSM strengthening applications is thus warranted.

2.5 Sustained Load

Sustained loads on structures produce time dependant increases in strain (creep strains) and in some cases corresponding time dependant decreases in stress (relaxation). The deformation which occurs at the instant that a load is applied to a concrete member is elastic, and the subsequent increases in deformation under sustained load are due to creep (Neville, 1996).
Because, depending on the magnitude of the sustained load, this strain can be several times larger than the strain generated during the initial loading, creep is of considerable importance to structural engineers. If the sustained load is removed from the concrete at a later time, after some creep has occurred, the strain immediately decreases by an elastic strain, which is generally less than the elastic strain which was generated at the time of loading. The subsequent gradual decrease in strain is known as creep recovery. Not all of the strain is recoverable, and the remaining strain is called residual deformation (Neville, 1995). Creep in concrete is caused by the rearrangement of calcium-silicate-hydrates (the building blocks of cement paste) at the nano-scale (Brehm, 2009). Therefore, creep is a function of the volumetric content of the cement paste in the concrete (Neville, 1995). The aggregate within the concrete acts as the primary restraint to creep. Concrete made with higher modulus aggregates therefore undergo smaller creep deformation under load. Research by Oldershaw (2008) has shown that beams subject to a period of sustained load prior to testing displayed greater stiffness than control beams, likely a result of the stiffening effect on concrete which experiences creep (Neville, 1995). A simple figure showing elastic strain, creep strain, elastic recovery, creep recovery, and residual deformation can be seen in Figure 2.11.

Creep is not a phenomenon limited only to concrete, it also occurs in FRP composite materials and adhesives. The creep response of FRPs is linked to:

- the type of polymer matrix and its stress history;
- the direction, type, and volume fraction of the fibre reinforcement;
- the nature of the applied loading; and
- the temperature and moisture conditions (Hollaway and Leeming, 1999).
Ceroni et al. (2006) studied the axial creep behaviour of glass, aramid, and carbon FRPs, and suggested 50 year creep rupture failure strengths of 29-55%, 47-66%, and 79-93% of respective ultimate strains. For the purpose of NSM strengthening, all of the fibres are in the same direction. The greater the volume ratio of fibres within the composite, the greater the stiffness, strength, and resistance to creep strains (Hollaway and Leeming, 1999). Part of this thesis investigates the performance of an NSM FRP strengthening system after a period of sustained load with and without exposure to freeze-thaw cycles. The sustained load was chosen such that the magnitude of the sustained load would be similar to the likely upper amount of load which would be experienced in a realistic strengthening scenario. This is discussed in Chapter 3.

2.6 Existing NSM Guidelines

Research in the area of NSM FRP strengthening for reinforced concrete structures has now advanced to the point where design guidelines are available in many regions (ACI, 2008; CSA, 2002; CSA 2006; Standards Australia, 2008). The following section summarizes some of the currently available guidelines. It should be noted that none of the guidelines specifically mention freeze-thaw cycling or freeze-thaw durability of NSM FRP systems.

2.6.1 ACI 440.2R-08

The ACI 440.2R-08 document (ACI, 2008), entitled Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, is the most thorough of the available design guidelines for NSM FRP strengthening of concrete. Flexural strength, shear strength, and development length calculations are all provided for NSM FRP systems for strengthening concrete. This guideline considers serviceability limit states (deflections and crack widths) as well as ultimate limit states (member failure, stress rupture, and fatigue) in design.
In terms of flexural strengthening, the document recognizes that the existing structure which is to be strengthened must be able to withstand a reasonable amount of load in the event the FRP system is damaged. This idea is incorporated into the design guidelines by stating the factored strength of the existing (unstrengthened) structure must be able to carry 110% of the specified design dead load and 75% of the specified design live load of the strengthened structure. 75% of the design live load was chosen on the basis that the statistical mean of yearly maximum live load on a structure is 50% of the design live load. In addition to ensuring that the new, strengthened member can still maintain structural integrity in the event that the FRP system is damaged, it is also critical to ensure that the initial failure mode of the member remains the same failure mode after strengthening. For example, it is necessary to check that a beam strengthened in flexure still meets shear design requirements such that it will not fail in shear once strengthened in flexure with FRP.

It is important to recognize that the FRP material properties provided by the manufacturer are initial values determined under ideal conditions in the lab. One of the main focuses of this research program is to determine the impact, if any, of freeze-thaw cycling on the performance of NSM FRP systems. Certain environmental exposures can potentially degrade the performance of FRP systems over time including: alkalinity, salt water, chemicals, ultraviolet light, high temperatures, high humidity, and freezing-thaw cycles. To account for long term characteristics in design, an environmental reduction factor is applied by ACI 440 to the initial FRP data provided by the manufacturer. Various reduction factors exist to account for the severity of the environment in which the FRP will be installed, and the type of fibres used. In general, carbon fibres are considered more resistant to environmental degradation than aramid fibres or glass fibres. Fibres composed of glass are most impacted by environmental conditions. The
environmental reduction factor lowers the ultimate stress and strain allowed in the FRP for design purposes.

The ACI design guidelines highlight several potential failure modes for concrete flexural members strengthened with FRPs. These are:

- crushing of the concrete in compression before yielding of the reinforcing steel;
- yielding of the steel in tension followed by rupture of the FRP laminate;
- yielding of the steel in tension followed by concrete crushing;
- shear/tension delamination of the concrete cover (cover delamination); and
- debonding of the FRP from the concrete substrate (FRP debonding, by one of a number of potential mechanisms as discussed previously).

To help improve the reliability of strength calculations, ACI 440.2R-08 uses two strength reduction factors. The FRP reduction factor, \( \psi_f \), is essentially an arbitrary reduction factor used to reduce the FRP contribution to moment capacity. The strength reduction factor, \( \Phi \), is used to reduce the strength of the entire section based on the amount of ductility at failure. For example, a strength reduction factor of 0.9 is applied when the strain in the steel is in excess of 0.005 at failure, while a strength reduction factor of 0.65 is used in settings where the internal reinforcing steel has not yet yielded at failure (and therefore provides less deflections and warning prior to failure). Linear interpolation of this value is used when the strain in the steel is between \( \epsilon_y \) and 0.005 at ultimate conditions.

The FRP strain at which debonding occurs, specifically for NSM FRP strengthening applications, has not yet been characterized sufficiently to implement rational design guidelines. However, the current draft of the code estimates FRP strain values between 60% and 90% of the ultimate strain of the FRP at debonding, depending on the member dimensions, steel and FRP
reinforcement ratios, and surface roughness of the FRP. Provided the bond length used in design is greater than the development length for the FRP, the guide allows a value of strain corresponding to 70% of the manufacturer’s guaranteed ultimate strength to be used as the debonding strain for NSM systems in the absence of specific debonding data. In shear strengthening situations, the maximum strain is not proportional to the ultimate strain of the FRP material, but rather 0.4%. This value was set to maintain aggregate interlock across shear cracks.

Finally, the sustained stress limits in FRP must be limited to lower values than will cause creep rupture of the fibres under long-term sustained loading conditions. Research has suggested that glass, aramid and carbon fibres can maintain 0.3, 0.5 and 0.9 times their ultimate strengths for sustained periods of 50 years. However, to minimize the probably of creep rupture failures, the guide recommends limiting the stresses to 0.2, 0.3 and 0.55 times the ultimate stresses for the respective fibres. Freeze-thaw cycling is not explicitly considered.

The ACI 440.2R-08 guidelines (ACI, 2008) were used to calculate the moment capacity of the strengthened slab-strips used in the current study. These calculations are summarized in Appendix A.

2.6.2 CSA S806-02

CSA-S806-02 (CSA, 2002), Design and Construction of Building Components with Fibre-Reinforced Polymers, is the Canadian Standard which governs the design of building components which contain FRPs. The code covers design with FRPs (both prestressed and non prestressed), development length calculations, and strengthening with surface bonded FRP. NSM strengthening is not specifically addressed in this document, although the approach used for externally bonded FRPs can be extended to treat the NSM case. The surface bonded FRP covers flexural strengthening, shear strengthening, column strengthening, as well as concrete and
masonry wall strengthening. The code uses a resistance factor for the FRP to reduce the FRP contribution to member strength (similar to resistance factors used in reinforced concrete design in Canada). The FRP resistance factor, $\Phi_f$, is taken as 0.75. Using strain compatibility and the assumption that plane sections remain plane, the code limits FRP strain to 0.7% in flexural strengthening applications, and 0.4% in shear strengthening applications. The lower bound for shear scenarios is used to reduce the probability of losing aggregate interlock at shear cracks. Freeze-thaw cycling is not explicitly considered.

### 2.6.3 CSA S6-06

The Canadian Highway Bridge Design Code, CSA S6-06 (CSA, 2006), discusses the use of FRPs in new construction as well as in reinforced concrete and timber bridge rehabilitation. The document discusses different fibre and matrix combinations in various conditions such as exposure to: acids, alkalis, high temperature, UV radiation, organic solvents, and ozone. Research is highlighted which explains that variability in tensile strength after the wet lay-up process shows much more variability than the variability in FRP bars or strips after environmental exposure. To account for this variation, the FRP resistance factor is the product of a material resistance factor and application factor. The material resistance factors for pultruded aramid, glass, and carbon FRPs are 0.60, 0.65, and 0.75 respectively. These values, however, are reduced by 25% when the fibres are applied using the externally bonded wet lay-up technique. Similar to other codes, the ultimate service strain for glass, aramid, and carbon fibres are 25%, 35% and 65% of the respective ultimate strains. Freeze-thaw cycling is not explicitly considered.

### 2.6.4 HB305-2008

HB305-2008 (Standards Australia, 2008) is the Australian Design Handbook for RC Structures Retrofitted with FRP and Metal Plates: Beams and Slabs. The handbook, published
by Standards Australia, covers debonding mechanisms in great detail. The same environmental reductions factors and creep rupture stress limits as suggested by ACI 440.2R-08 (ACI, 2008) are used. The handbook allows for any method of design, provided that all debonding mechanisms and their interactions can be accounted for. The document identifies five modes of plate debonding: intermediate crack (IC) debonding, flexural intermediate crack (FIC) debonding, critical diagonal crack (CDC) debonding, plate end (PE) debonding, and VAy debonding. The various modes of debonding are represented in Figure 2.13. Although FIC debonding is not labelled, it is the specific case where IC debonding is initiated due to a flexural crack. A generic IC debonding model (Seracino et al., 2007a) was used to predict the debonding loads for the slab strips tested in this thesis. The results of this model are discussed in Chapter 5.

2.7 Summary

The deterioration of infrastructure in Canada and the rest of the industrialized world is a major concern to the engineering community. Of the bridges in Quebec, 49% are deficient, as are 32% of the bridges in Ontario. There is therefore an urgent need for better design, construction, inspection, and repair of Canadian infrastructure. One potential solution to these problems is strengthening using FRPs. FRPs can be used to increase the shear capacity, flexural capacity, and axial capacity of reinforced concrete members. A recently developed FRP strengthening technique involves installation of the FRP in the near surface of the member. Near surface mounted FRP reinforcement has numerous advantages over externally bonded FRP strengthening. This method is particularly attractive for negative moment strengthening and it provides greater ultimate capacity and greater deformation capacity when compared to externally bonded systems. The method also mitigates bond failures and allows a greater proportion of the FRPs strength to be utilized prior to failure. Early work examining NSM FRP used square or
round rods. NSM strips, however, have been shown to be less susceptible to debonding. The American Concrete Institute and others have produced recommendations for NSM FRP strengthening scenarios. Models now exist which can be used predict the intermediate crack induced debonding stress for NSM systems. Although full-scale ‘real world’ applications of strengthening using NSM FRP have been successfully implemented, uncertainty remains around the cold-regions durability of NSM FRP. Specifically, the freeze-thaw durability of NSM FRP has yet to be examined.

The following chapter highlights the experimental procedure used to examine both the freeze-thaw and sustained load durability of a commonly available NSM carbon FRP strip strengthening system for flexural or shear strengthening of reinforced concrete flexural members.
Figure 2.1: Modified pull-out test setup designed by De Lorenzis et al. (2002) to investigate the bond performance of NSM FRP rods in concrete (De Lorenzis et al., 2002)

Figure 2.2: Cohesive shear failure in the concrete (left), and tensile splitting of the adhesive (right) observed in modified pull-out bond testing of NSM FRP rods (De Lorenzis et al., 2002)
Figure 2.3: Normal bond stress distribution around a rectangular NSM strip (left) and a circular NSM rod (right) showing how the bond stresses around a rod create forces which push the rod out of the adhesive.

Figure 2.4: Installing NSM carbon FRP rods on the soffit of a bridge deck near Rolla, Missouri (Alkhrdaji et al., 1999)
Figure 2.5: Cutting grooves prior to installation of NSM FRP bars to strengthen a reinforced concrete silo (Prota et al., 2001)

Figure 2.6: Parameters and dimensions used in the Seracino et al. (2007a) models for bonded rectangular FRP plates and strips. The aspect ratio is the ratio of the depth of the failure plane to the width of the failure plane. Externally bonded plates have a small aspect ratio (left) and near-surface mounted plates have a larger ratio (right) (Seracino et al., 2007a)
Figure 2.7: non-air-entrained concrete (left), and air-entrained concrete (right) showing uniformly distributed air voids throughout the paste of the concrete with a common pin for reference (http://www.cement.org/tech/cct_dur_freeze-thaw.asp)

<table>
<thead>
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<th>Matrix</th>
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<th>Surface</th>
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</thead>
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<td>Epoxy/vinylester</td>
<td>8.26</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>Carbon</td>
<td>Epoxy/vinylester</td>
<td>8.00</td>
<td></td>
</tr>
<tr>
<td>C3</td>
<td>Carbon</td>
<td>Epoxy</td>
<td>7.94</td>
<td></td>
</tr>
<tr>
<td>G1 (prototype)</td>
<td>Glass E</td>
<td>Thermoplastic</td>
<td>12.00</td>
<td></td>
</tr>
<tr>
<td>G2</td>
<td>Glass E</td>
<td>Polyester</td>
<td>6.35</td>
<td></td>
</tr>
</tbody>
</table>

* Obtained as average values after 12 measurements for each rod.

Figure 2.8: Summary of FRP rods examined in the durability investigation performed by Micelli (2004)
Figure 2.9: ASTM D445 test setup used in the durability study by Micelli (2004)

Figure 2.10: Schematic of the environmental exposures used in the durability study by Micelli (2004)
Figure 2.11: Schematic showing simplified creep and creep recovery for concrete under uniaxial stress (Bisby, 2006)

Figure 2.12: Schematic showing the intermediate crack (IC) induced debonding mechanism for externally bonded FRPs bonded to concrete substrates (Seracino et al., 2007)
2.13: Various potential debonding mechanisms for FRP strengthened reinforced concrete beams and slabs (Oehlers, 2005)
Chapter 3

Experimental Procedure

3.1 Introduction

The literature review presented in Chapter 2 highlighted the lack of durability data currently available on the performance of near surface mounted FRPs in cold climates. Specifically, the performance of NSM systems after exposure to repeated freeze-thaw cycles has not been investigated. In this chapter, the experimental program performed for the current thesis, consisting of 24 medium scale NSM strengthened steel reinforced concrete slab-strips (see Figure 3.1) and 16 NSM direct pull-out bond tests (see Figure 3.20), is discussed. In both cases the FRP used for the NSM application was Aslan 500™ CFRP tape, manufactured and supplied by Hughes Brothers Inc, USA. The manufacturer-specified mechanical properties of Aslan 500 CFRP are summarized in Table 3.1.

The testing program discussed in this thesis was specifically chosen to investigate the performance of NSM FRPs in a climate where the potential damage from cold environments and freeze-thaw cycling could cause deterioration of structural performance, as would be experienced in an application with an exterior exposure in a cold climate. This thesis presents testing which has bearing on both the shear and flexural strengthening performance of this specific NSM carbon/vinylester FRP tape system for concrete structural members. Two distinct adhesives were used: (1) a crack injection epoxy resin and (2) a high-strength non-shrink cementitious grout. The adhesives were chosen through discussions with an industry partner (Vector Construction, Winnipeg, MB). Both adhesives can be applied under pressure using crack injection equipment.
This allows the FRP to be applied into a dry groove of smaller dimensions than would be feasible without pressure injection. Once installed, tape is applied over the groove to seal it, and the adhesive is then pumped in. The pressure injection not only forces the adhesive all around the FRP in the smaller groove, it concurrently fills all cracks and voids in the concrete substrate which are directly connected to the groove, thereby further strengthening the member. Although previous research has suggested that epoxy adhesives outperform cementitious grouts in NSM applications at room temperatures, enabling a higher proportion of the strength of the FRP tape to be utilised before experiencing bond failure (Burke, 2008), the grout was included in the current study to study the possibility that the similar thermal expansion of the grout, as compared to the concrete substrate, might lead to less adhesive-concrete bond degradation during thermal changes and therefore to superior retention of mechanical and bond performance (compared to the epoxy adhesive) after exposure to repeated freeze-thaw cycles. The cementitious grout is, however, advantageous in terms of its reduced environmental hazards during handling, much lower cost compared to the epoxy, and a finished appearance which is virtually seamless with the rest of the concrete structure.

For the slab-strip portion of the testing program, the two adhesive systems were each examined for their performance after exposure to one of four conditioning regimes, including:

1. ambient conditions in the structural engineering laboratory (room temperature controls);
2. ambient conditions under sustained load at room temperature (sustained load controls);
3. 300 freeze-thaw cycles in the civil engineering environmental chamber without any sustained load; and
4. 300 freeze-thaw cycles while under sustained load.
The above different series of tests were performed so that it was possible to distinguish between the effects of sustained load versus freeze-thaw cycling, in the event that any deterioration in performance was observed. A sustained load setup developed previously by Oldershaw (2008) was used to apply flexural loading to the sustained load slab strip specimens (see Figure 3.14 and Figure 3.15). The duration of sustained loading was chosen as the time required to complete 300 freeze-thaw cycles (approximately 120 days). After conditioning of the slab-strips was completed they were tested to failure in four-point bending under a displacement control mode, as described below.

A series of single-lap bond pull-out tests was performed in addition to the flexural tests. This was done because, as noted by previous researchers, “the debonding failures observed in flexural tests differ greatly from those found in bond tests, as debonding in a beam is related primarily to the concrete cracking in the cover region rather than pull-out failure along the NSM strip-adhesive interface” (Teng et al., 2006). Thus, by conducting both flexural and bond tests, both aspects of bond behaviour were studied. It is also worth noting that the bond pull-out tests are reasonable representations of NSM strips acting in concrete shear strengthening applications. Similar to the slab strip specimens, the direct pull-outs were also tested under displacement controlled loading. The same two adhesives (epoxy and cementitious grout) as were used in the flexural tests were also studied in the pull-out tests. The two exposure conditions examined were:

1. ambient conditions in the structural engineering laboratory (room temperature controls); and

2. exposure to 150 freeze-thaw cycles.

Due to time constraints in performing large numbers of freeze-thaw cycles and limited space with the freeze-thaw chamber at Queen’s University, for the pull-out specimens this was
reduced to 150 cycles. Further, it was not possible to apply sustained loads to the pullout specimens during freeze-thaw cycling, and the effects of sustained load have not been investigated in this context.

The following sections present and discuss the design, fabrication, strengthening, conditioning, and testing procedures used for the two types of tests discussed above (flexural and bond tests).

### 3.2 Slab Strip Specimens

#### 3.2.1 Design

The slab strip specimens were designed to represent a scaled down one way reinforced concrete slab based on prior research by Burke (2008) which examined similar specimens at cold temperatures or during heating to 200°C. The details of the slab strips are given in Figure 3.2. The design of the slab strips was performed such that the strain in the FRP could approach its ultimate strain prior to failure. The specimen was designed so that the bond of the NSM FRP would fail prior to concrete crushing in the compression zone or shear failure of the member, but after yielding of the internal steel reinforcement. The design calculations for the slab strip specimens are summarized in Appendix A. The codes and guidelines used during design of the slab strips were: CSA A23.3-04 (CSA 2006), ISIS’ Education Modules (ISIS 2003), ACI 318-05 (ACI 2005), and ACI 440.2R-08 (ACI 2008). Although both of the above FRP design guidelines suggested limiting the strain in flexural FRP to 70% of the ultimate FRP strain, to maximize the probability of observing a bond failure in the laboratory the design of the slab strips used $f_{adv}$ as the FRP design strain.

The dimensions of the slab strips were chosen to give as realistic proportions as possible while minimizing their overall weight since the testing was performed in a room without
overhead crane access. Note that the dimensions quoted below represent hard conversions from US customary units. The width of the section (254 mm) was selected to be large enough so that the edge distances for the NSM FRP strengthening would be greater than the minimum edge distance requirements of four times the depth of the NSM groove, as suggested by ACI 440.2R-08 (ACI 2008). The depth of the slab (102mm) was selected as a reasonable lower-bound for a real world strengthening situation. The length of the specimens (1524 mm) was selected to attain a realistic shear span to depth ratio (4.8) so as to minimize arching action during testing. In general, the specimens were designed to be as large as possible while still allowing them to be tested in a room without an overhead crane to move them in and out of the freeze thaw tanks and the testing frame.

The internal flexural steel used in the specimens consisted of two D5 deformed steel bars placed 127 mm apart from centre to centre. The steel bars had 25 mm of clear cover to the tension face of the member. The flexural steel reinforcement ratio (0.34%), while comparatively low, was greater than the minimum required steel reinforcement ratio (0.002\(A_g\)) given by A23.3-04 (CSA 2006). The low reinforcement ratio was chosen so as to represent a deficient slab which might require flexural upgrading. The specimen design did not call for any additional shear reinforcement.

A 25 mm by 25 mm square notch was cast across the tensile face of the specimen at midspan to allow a strain gauge to be applied directly to the NSM FRP strip within the constant moment region (shown in Figure 3.1). The notch also allowed for direct visual access and the use of high resolution digital camera to calculate strains in the FRP in the event that one of the strain gauges was damaged during fabrication, freeze-thaw cycling, or testing.

3.2.2 Fabrication
The fabrication of the slab strip specimens was performed in the Structural Engineering Laboratory at Queen’s University. The specimens’ formwork was constructed using 1220 mm by 2440 mm sheets of 19 mm thick plywood. A photo of the formwork just prior to casting the concrete is shown in Figure 3.3. The notch at midspan was formed using a wooden block. Similar blocks were also used at each end of the beam to secure the internal steel reinforcing bars which were held in place using fencing staples. The staples were used to prevent movement of the reinforcing bars by accidental contact with the vibrating wand or from ‘floating’ during vibration. The D5 deformed steel bars used in the specimens were shipped and received in a coil. Curved lengths of the bar were cut, straightened, and then re-cut to the exact required length prior to installation in the formwork.

The concrete was cast in the Structural Engineering Laboratory using concrete supplied by a local concrete ready-mix company. The air-entrained mix was specified to have a 28-day compressive strength of 35 MPa with a slump between 75 mm and 100 mm. Tests were performed prior to placing the concrete to ensure that the air entrainment and slump were satisfactory. Following placement of the concrete using an overhead hopper, all of the slab strips were vibrated using an electric wand (Figure 3.4) and given a smooth trowelled finish. The specimens were covered with plastic drop sheet and allowed to moist cure for three days before being stripped from the formwork and stored in ambient conditions in the lab. Unfortunately, one of the slab strips was cracked into two pieces while being stored in the lab prior to strengthening. The slab strip testing program was thus accidentally reduced from 24 specimens to 23.

3.2.3 FRP Strengthening System

All of the slab strips were strengthened in flexure with a single strip of Aslan 500 #2 CFRP tape, which was supplied by Hughes Brothers Inc (USA). Two identical unstrengthened
slab strips (cast from a different concrete pour but with a similar concrete compressive strength) were tested by Burke (2008) and have been included as benchmark specimens in the current study.

The FRP was placed into a groove cut along the slab strips’ tensile faces as described below. The Aslan 500 #2 CFRP tape was the smallest strip available for this type of application. Ideally, a smaller strip would have been used as the level of strengthening was much larger than anything which would be attempted in practice. The strengthening calculations summarized in Appendix A indicate that ACI 318-05 (ACI 2005) predicted the unstrengthened slab strip to have an unfactored ultimate moment capacity of 3.06 kN·m, while ACI 440.2R-08 (ACI 2008) predicted the strengthened slab strip to have an unfactored ultimate moment capacity of 6.99 kN·m. Although practically speaking this is an unreasonable level of strengthening, the section could not be made any larger due to the lack of crane access in the testing rooms.

The NSM grooves were cut into the tensile faces of the slab strips using a tuckpoint grinder with a diamond blade (Figure 3.5). The dimensions of the groove, dictated by the dimensions of the available diamond blades, were 6.4 mm wide by 21 mm deep. After grinding the groove, the surface was consistent but rough enough to provide a sound surface for the bond of the adhesive without any additional surface treatment, shown previously by De Lorenzis et al. (2004). An aluminum guide was specially designed to guide the tuckpoint grinder along a straight line while keeping the blade perpendicular to the surface of the tensile face of the slab strip. Images of the guide installed on one of the slab strips are shown in Figure 3.6, while Figure 3.7 shows one of the grooves being cut. Approximately 25 mm of concrete was left uncut at each end of the slab strip to prevent adhesive from running out of the groove during application of the NSM strips.
The NSM CFRP tape was shipped in a continuous coil. Once the FRP tape arrived in the laboratory, it was carefully cut into 1397 mm long sections using a hacksaw. Prior to installing the FRP in the NSM groove, a bead of caulking and a foam strip were laid into the groove on either side of the notch to provide a seal and prevent the adhesive from flowing out of the groove into the notch at midspan (see Figure 3.9). The adhesive was placed such that 25 mm of FRP was unbonded on either side of the notch.

Once the caulking had set, the adhesive was poured into the groove. The epoxy adhesive, Kemko 038, is a two-part adhesive composed of a clear resin and a dark purple hardener. The manufacturer-specified properties of the epoxy are summarized in Table 3.2. The two parts were mixed equally by volume, and then hand stirred for five minutes as per the manufacturer’s instructions. Using small paper cups, the adhesive was then carefully poured into the NSM groove where it flowed around the FRP strip until the groove was filled. Careful attention was paid to ensure the slab strips were level prior to pouring the adhesive to prevent it from accumulating at the low end. The epoxy was then allowed to cure for several days before the slab strips were carefully stacked. Of the 23 slab strips, 11 were strengthened using the Kemko 038 epoxy. Two of the slab-strips strengthened with epoxy adhesive did not cure properly and had to be removed from the testing program. It is believed that this occurred as a result of a momentary lapse of concentration during fabrication which resulted in reversing the mix volumes of the resin and hardener. Neither a heat lamp nor additional hardener solved the problem of the un-cured epoxy. The slab strip testing program was therefore further reduced from 23 to 21 specimens. It should be noted that the slab strips were strengthened upside-down to make the strengthening work easier. In reality, it would not be possible to strengthen using this method in positive bending moment regions (sagging regions). In a real field application, where strengthening
operations would need to be performed overhead, it is envisioned that the NSM grooves would be filled using crack injection techniques (this is precisely why crack injection epoxy and grout have been used in the current study).

Similar to the epoxy, the Target 1118 grout used for the cementitious adhesive was also a crack injection product and also had a very low viscosity which required caulking and foam strips to prevent the adhesive from running out of the groove into the notch at midspan. The properties of Target 1118, as supplied by the manufacturer, are listed in Table 3.3. Of the 21 slab strips, 12 were strengthened using the Target 118 cementitious grout. The grooves were thoroughly wetted prior to adding the grout to prevent additional water from being drawn out of the grout and into the surrounding concrete. The grout, which was shipped in powder form, was mixed with 365 mL/kg of powder using a shear paddle mixer and a hand drill. Like the epoxy, the grout was carefully poured into the grooves using small Dixie cups. Once the grout was poured into the groove, the slab strips were covered with plastic drop sheet and wet cured for three days. The drop sheet was briefly removed once a day to re-wet the grout and prevent cracking and shrinkage of the adhesive.

3.2.4 Test Setup

After being strengthened and subjected to the requisite environmental conditioning programme, the slab strips were tested to failure in four point bending, as shown in Figure 3.1. The loading regime had a constant moment span of 432 mm, with two shear spans of 508 mm. The test setup was designed previously by Burke (2008). The slab strips were tested upside down, with the tensile face pointing up to allow a clear view of the NSM bond line throughout the tests. Of the four load reaction points, three were rollers and one of the middle supports was fixed horizontally to act as a pin. A cylindrical steel roller was used at each reaction point, with a
steel bearing plate to prevent localized crushing of the concrete under the rollers. The steel plates on the bottom two (middle two) supports were 102 mm wide, while the plates on the top two supports were 38 mm wide. Load was applied under constant actuator displacement rate of 2 mm/min using a 220 kN hydraulic ram within a cold temperature structural testing room at Queen’s University (although the tests were performed at room temperature). The loading rate was chosen to allow the test to run over a long enough period of time to attain a reasonable amount of data. Furthermore, the test was run over a long enough period of time that strain rate dependencies would not be an issue.

3.2.5 Instrumentation

The instrumentation used during flexural testing of the slab-strips included: linear potentiometers, Pi Gauges, and bonded foil strain gauges. A high resolution digital camera was also used to take photos at five second intervals during testing. Initially, it was planned that these photos might be used to calculate strains and curvatures throughout the tests. However, it was decided that the traditional methods of data collection would provide sufficient data and the additional step of image analysis was not performed. Additionally, thermocouples were installed on one of the slab-strips which was monitored as it underwent freeze-thaw cycling to accurately determine the temperatures at certain key locations within the member during the freeze-thaw cycles.

Linear potentiometers (LPs) were used to calculate the vertical movements of the slab-strips during testing. One LP was placed directly under the centre of the specimen, and one was placed on the loading plate at each end. By averaging the two downward displacements of the end LPs, and adding the upward displacement of the centre LP, the total vertical displacement of the slab-strip was continuously monitored through the tests. The LPs were calibrated using a
specially designed digital calibration tool consisting of a digital micrometer on a machined base with an accuracy of 1 µm. Each LP was calibrated in the specific direction it would be moving during testing. The calibration curve (linear) was calculated using 10 calibration points for each LP.

Three Pi Gauges were used to measure strains and calculate curvatures over the height of the slab-strip at midspan. A Pi Gauge, shown in Figure 3.8, is a simple combination of strain gauges and an arch-shaped spring plate. The gauge is designed to measure strains and crack openings in concrete over the gauge length of the transducer. Pi Gauges, however, can be used in any situation to measure small displacements. The gauges were placed on the side of the slab strips using five-minute epoxy on the heads of steel bolts placed through the gauges. Careful attention was paid to the placement of the Pi Gauge bolts to ensure the Pi Gauge would be installed at the desired location with the appropriate gauge length it was designed for. The top Pi Gauge was placed at the same height as the centre of the FRP strip, 11 mm from the top edge of the slab strip. The middle Pi Gauge was placed at the centre height of the cross-section. The bottom Pi Gauge was placed 11 mm from the bottom edge of the concrete. This setup was chosen to allow the strain profile to be calculated over the height of the cross-section, and therefore to enable calculation of the curvatures during testing. Calibration of the Pi Gauges was performed with the same digital calibration tool as was used for the LPs. The top two Pi Gauges were calibrated in tension, and the bottom Pi Gauge was calibrated in compression.

A single Vishay SR-4 250LW 120Ω general purpose unidirectional bonded foil strain gauge was applied to the centre of each NSM FRP strip at midspan. The foil strain gauges were installed according to Vishay Micro-Measurements Instruction Bulletin B-137-16 (Vishay, 2005). In order to protect the strain gauges during the slab strips’ exposure to freeze-thaw cycles in the
environmental chamber, a small quantity of marine grade silicone was placed around each gauge after its application (Figure 3.11) to seal the gauge and prevent the freeze-thaw action from damaging it or its bond to the FRP strip. In addition, the lead wires of the strain gauges were placed in small rubber tubes which were anchored into the wet silicone, effectively sealing the entire strain gauge system from water during freeze-thaw cycling. A schematic showing the dimensions of the loading setup and the locations of the instrumentation is shown in Figure 3.12.

Type-T thermocouples were used to measure the temperatures within the slab strips during the freeze-thaw cycling. One row of three thermocouples was placed at the centre of a concrete cross-section at one quarter of the length of a slab strip. Additionally, two previously tested specimens (one strengthened with epoxy and the other with cementitious grout) from a previous testing program (Burke, 2008) were used as dummy specimens with one thermocouple placed at the FRP-adhesive interface for each of the adhesives examined. These thermocouples were used in dummy specimens to prevent the artificial corruption of the bond line in the actual tested specimens. Finally, one thermocouple was used to record the temperature of the air in the environmental chamber. The temperatures recorded from these thermocouples can be seen in Figure 3.13. All thermocouples were placed at a location one quarter of the length of the slab strips (half way between the notch and the end of the slab-strip). The thermocouples which were installed in the concrete can be seen in Figure 3.10 prior to casting the concrete.

3.2.6 Freeze-Thaw Cycling

The slab-strips were exposed to 300 freeze-thaw cycles in a large-scale environmental chamber at Queen’s University. The number of cycles was essentially arbitrary, but was chosen to match the numbers of freeze-thaw cycles performed by previous researchers examining the effects of freeze-thaw cycles on FRP strengthened reinforced concrete members (e.g., Kong,
Based on historical temperature records, this number of cycles could represent anywhere from 10 to 20 years of exposure for an exterior application in Toronto (Bisby, 2002). The nature of the cycles, however, was more severe than would be typically encountered in reality. The slab strips were subject to their freeze-thaw cycles inside a large thermal chamber (cold room), where they were placed inside large metal tanks. First, the temperature in the cold room was cooled to -30°C over a period of 5 hours. The air temperature was then ramped up to 20°C. During reheating, as soon as the temperature in the room was above freezing, pumps filled the tubs containing the slab strips with warm water. This provided an extreme thermal shock to the specimens as well as a near instant thaw. After 70 minutes of thawing, the pumps turned off, gravity drained the water out of the tanks, and the room temperature was once again dropped to -30°C. The entire cycle can be seen graphically in Figure 3.13. Photos of the slab strips under sustained load during the freezing phase and thawing phase are contrasted in Figure 3.16. Ten of the slab-strip specimens were exposed to freeze-thaw cycles in the environmental chamber. Five of these ten (three with grout adhesive and two with epoxy adhesive) were placed in the tanks without sustained load, and five (three with grout adhesive and two with epoxy adhesive) were exposed to freeze-thaw cycles while under sustained load. The sustained loading rig is described in the following section.

### 3.2.7 Sustained Load

The sustained load setup used in this testing program was designed previously by Oldershaw (2008). The beams were loaded two at a time in back-to-back, self reacting frames. The setup consisted of a hydraulic ram with a hand pump, a load cell, six small steel hollow structural sections (HSS), two large HSS sections, four threaded rods, and 12 nuts. The setup can be seen during load application in Figure 3.14, while Figure 3.15 shows the loading setup after
the load has been applied and the hydraulic ram has been removed. The loading was applied in
the same configuration (shear spans of 508 mm and a constant moment span of 432 mm) as used
in the test setup that was used to fail the beams (described previously). The hydraulic ram was
used in conjunction with a load cell with a digital display to apply the load to the slab-strips.
Once the load was applied to the pair of slab strips, nuts were used to lock the load below the
hydraulic ram, and the ram could then be moved and used to apply load to the next set of slab
strips. Due to relaxation of the sustained load test setup as well as creep of the strengthened
concrete slabs, the sustained load was re-applied four times throughout the sustained load period.

The magnitude of sustained load applied to the slab strips was chosen as 15 kN. This
load was chosen because it was in excess of the ultimate loads reached by identical
unstrengthened slab-strips by roughly 25%, and would thus be considered a reasonable level of
strengthening that might be attempted in practice. Furthermore, the lowest ultimate load
observed by Burke (2008) during his testing of similar slab-strips was 23 kN. To minimize the
risk of sudden failure while under sustained load, a reasonable cushion of additional strength was
left unused while under sustained load. Based on Burke’s prior tests on identical beams, it was
expected that the strain in the NSM FRP strips would be in the range of 0.4 % under the sustained
load of 15 kN. A numerical layer model, discussed in detail in Chapter 5, predicted a strain of
0.52% for a sustained load of 15 kN. Six of the slab strips (three with the cementitious grout
adhesive and three with the epoxy adhesive) were exposed to sustained load and left in ambient
conditions in the lab. Five of the slab-strips (three with cementitious grout and two with epoxy)
were exposed to freeze-thaw cycles while under sustained load. Because there were an uneven
number of slab-strips in the environmental chamber, a large HSS was used to react against the
final slab strip which did not have a partner slab strip to react against.
3.2.8 Testing Procedure

Load was applied under displacement control at a rate of 2 mm/min using a 220 kN hydraulic ram installed in a self-reacting testing frame. Each test was continued until failure, which was sudden and violent in all cases as described in detail in Chapter 4.

3.3 Pull-Out Specimens

3.3.1 Design

The modified pull-out specimens used in the testing regime described in this thesis were designed according to principles suggested by De Lorenzis et al. (2002). A dimensioned 3-D computer graphic of one of the test specimens with the FRP installed can be seen in Figure 3.17. The test method involved installing an NSM FRP strip on the inside face of a C-shaped block of concrete. The design allowed the concrete to be restrained by nuts on the bottom of the four threaded rods while the FRP was pulled upwards until bond failure. The C-shaped concrete block was modified slightly to provide enough space to use the tuck-point grinder to cut the groove for embedment of the NSM FRP tape. A 3-D representation of the pull-out, as well as an actual specimen in the test setup prior to testing, are shown in Figure 3.20. The C-shaped concrete block is superior to square or rectangular bond test specimen bases which have been used by other researchers because:

- only one bond length is required. Previous bond test specimens used by some previous authors have used two bonded lengths which were pushed apart, which required putting the concrete into compression at the loaded end of the bond and therefore artificially increasing confinement around the most highly stressed portion of the bond;
- the bond can be easily monitored visually during testing; and
• the FRP is located in the centre of the four threaded reaction rods, thus minimizing the chances of accidental eccentricity of loading.

3.3.2 Fabrication

Fabrication of the modified pull-out specimens was performed in the Structural Engineering Laboratory at Queen’s University. The formwork was again constructed using 1220 mm by 2440 mm sheets of 19 mm thick plywood. A photo of the formwork just prior to concrete casting can be seen in Figure 3.19. The concrete was cast at the same time as the slab-strip specimens, using exactly the same concrete mix. Following the placement of the concrete using an overhead hopper, all pull-out specimens were consolidated using an electric wand vibrator (Figure 3.4) and given a smooth finish with a steel trowel. The specimens were then covered with plastic drop sheets and allowed to moist cure for three days before being stripped from the formwork and stored in ambient conditions in the lab. Anchor bolts were cast into each of the four corners of the C-shaped blocks to allow them to be anchored to the base of the testing frame during pullout testing.

3.3.3 FRP Strengthening System

One strip of Aslan 500 #2 CFRP tape was installed in each pull-out specimen. A total length of 914 mm of FRP was used for each pull-out specimen. This total length was necessary to allow the mechanical wedge action grips used in the testing room to reach the FRP. One end of the FRP strip was tabbed on both sides using Sikadur 30 epoxy adhesive and prefabricated GFRP tabs. The Sikadur 30 was mixed at 3:1 by mass of Part A and Part B, as per the directions provided by the manufacturer.

The FRP was placed into the groove cut along the inside face of the block (see Figure 3.17). The NSM grooves were cut using the same tuckpoint grinder with a diamond blade
(Figure 3.5) as was used for the slab-strips. The dimensions of the groove, dictated by the dimensions of the blade, were 6.4 mm wide by 21 mm deep. The aluminum guide was again used to guide the tuckpoint grinder along a straight line while keeping the blade perpendicular to the surface of concrete. The FRP was installed with a 152 mm bond length, starting 25 mm from the top of the pull-out specimen. This bond length was chosen because a bond model available in the literature (Seracino et al., 2007b) predicted that this bond length would produce failure strains slightly more than 60% of the manufacturer’s guaranteed ultimate FRP strain. This value was chosen as an upper bound to allow the development of large FRP strains while ensuring bond failure rather than FRP rupture. Prior to installing the FRP in the NSM groove, a bead of caulking and a foam strip were laid into the groove at both ends of the bond length to prevent the adhesive from flowing out of the groove (as was performed during fabrication of the slab strip specimens). Once the caulking had set the adhesive was poured into the groove. Of the 16 modified pull-out specimens, 8 were strengthened using Kemko 038 epoxy adhesive; the remaining 8 FRP strips were bonded using Target 1118 cementitious grout adhesive. Similar to the slab strip specimens, the NSM grooves were thoroughly wetted prior to adding the cementitious grout to minimize leaching of water from the grout into the surrounding concrete. After installation, the grout was covered with plastic drop sheet and moist cured for three days.

3.3.4 Test Setup

The test setup for the pull-out specimens can be seen in Figure 3.20. The aluminum cylinder in the photo was used only to support the weight of the hydraulic grips prior to testing. If the grips were not supported they drifted down when the hydraulic pump for the loading ram was not running. The threaded rods at each corner of the specimen were placed through and bolted to a metal base plate. Large steel C-clamps, shown in Figure 3.20, attached the steel base
plate to the bottom beam of the self reacting testing frame. One of the advantages of this setup was direct visual access to the bonded length, which allowed for high-resolution digital images of the loaded end of the bond to be captured during testing. The images were used to track the movement of the FRP strip (loaded end bond slip), just above the bonded length, with marginal success as described in Chapter 4.

### 3.3.5 Instrumentation

The instrumentation used during the modified pull-out tests was limited to a bonded foil strain gauge and a high resolution digital camera. Load and stroke of the hydraulic loading ram were also recorded during testing. The strain gauge was applied to the FRP strip just above the termination of the bonded length. A Pi Gauge was used for the first few tests in an attempt to measure loaded end bond slip between the FRP strip and the concrete block. However, cracking of the concrete around the loaded end of the bond lifted the Pi Gauge anchor point during testing and rendered the collected data essentially useless. Digital image analysis was used to monitor the loaded end bond-slip response for the specimens bonded with cementitious adhesive, but for specimens bonded with epoxy only the load versus stroke comparisons are available. In order to use the high-resolution digital image technique to track pixel movements and calculate strains and displacements, a speckled texture (shown in Figure 3.21) was applied to the FRP strip using white paint. Furthermore, a reference box was placed in each image with a fixed scale so that the pixel movements could eventually be correlated to distance and converted into displacements.

### 3.3.6 Testing Procedure

Load was applied under displacement control at a rate of 0.4 mm/min by a 220 kN hydraulic ram installed in a self reacting testing frame.
3.4 Ancillary Testing

Ancillary tests were performed on all the materials used in the fabrication of the NSM FRP strengthened reinforced concrete members. Concrete cylinders were cast at the time of specimen fabrication. These were tested for compressive strength at 7 and 28 days, as well as at the time of testing the slab strips and pull-out specimens. Four cylinders were additionally exposed to ambient conditions in the laboratory, and four were exposed to freeze-thaw cycling. Tests on the D5 wire used as the reinforcing steel were performed by Ranger (2007). Tests were performed by Burke (2008) to determine the stress-strain response of the Aslan 500 carbon FRP strip. The results of all tests, both main and ancillary, are presented in Chapter 4.

3.5 Summary

Near surface mounted (NSM) FRP reinforcement has been shown to be a promising technology for the strengthening of reinforced concrete structures in both shear and flexural settings. However, many issues remain with regards to the long term durability of this method. Specifically, the performance of NSM FRP in a climate where the potential damage from cold environments and freeze-thaw cycling (which could cause structural degradation) has yet to be examined. This chapter highlighted the experimental procedure of a research program which was developed to examine this knowledge gap. A two part procedure was developed. First, a series of 21 slab strips were constructed, strengthened, conditioned, and tested. The slab strips were exposed to one of four conditions: ambient conditions (room temperature controls), ambient conditions under sustained load (sustained load controls), 300 freeze-thaw cycles, or 300 freeze-thaw cycles under sustained load. Second, a series of 16 pull-out specimens were constructed, conditioned, and tested. The pull-out specimens were exposed to either ambient conditions (room temperature controls) or 150 freeze-thaw cycles. In both portions of the testing program, two
commonly available pressure injection adhesives were used for bonding the NSM FRP to the concrete. The results and analysis of the testing program are given in Chapters 4 and 5, respectively.
Table 3.1: Properties of Aslan 500, from the manufacturer’s website (www.hughesbros.com)

<table>
<thead>
<tr>
<th>Width (mm)</th>
<th>Thickness (mm)</th>
<th>Cross Sectional Area (mm²)</th>
<th>Tensile Strength (MPa)</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Ultimate Strain</th>
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<tbody>
<tr>
<td>16</td>
<td>2</td>
<td>31.2</td>
<td>2068*</td>
<td>124</td>
<td>0.017</td>
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* Tensile strength indicated is the average minus three standard deviations (ACI440/f<sub>u</sub> value)

Table 3.2: Properties of Kemko 038 Regular IR, taken from the manufacturer’s website

<table>
<thead>
<tr>
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<td></td>
<td>Weight</td>
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<tr>
<td>Color</td>
<td>Part A</td>
<td>Visual</td>
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<td></td>
<td>Part B</td>
<td>Dark Purple</td>
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<tr>
<td></td>
<td>Mixed</td>
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<tr>
<td>Weight per Gallon, lb</td>
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<tr>
<td></td>
<td>Part B</td>
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<td>Viscosity, cp</td>
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<td>Mixed Viscosity @ 50 F, cp</td>
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<td>Gel Time, 100 g, minutes: @ 73 F</td>
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<td>Flexural Modulus, psi</td>
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Table 3.3: Properties of Target 1118 Grout at 20 °C, taken from the manufacturer’s website

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<td>Expansion Before Setting (% volume)</td>
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<tr>
<td>Compressive Strength (MPa)</td>
<td></td>
</tr>
<tr>
<td>at 24 hours</td>
<td>26 36</td>
</tr>
<tr>
<td>at 72 hours</td>
<td>45 45</td>
</tr>
<tr>
<td>at 7 days</td>
<td>55 56</td>
</tr>
<tr>
<td>at 28 days</td>
<td>75 75</td>
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Table 3.4: Slab strip testing program

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<th>Specimen Name</th>
<th>Study Variable</th>
<th>Adhesive Type</th>
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<td>E-RT-1</td>
<td>NSM FRP Bond</td>
<td>Epoxy</td>
</tr>
<tr>
<td>2</td>
<td>E-RT-2</td>
<td>NSM FRP Bond</td>
<td>Epoxy</td>
</tr>
<tr>
<td>3</td>
<td>G-RT-1</td>
<td>NSM FRP Bond</td>
<td>Grout</td>
</tr>
<tr>
<td>4</td>
<td>G-RT-2</td>
<td>NSM FRP Bond</td>
<td>Grout</td>
</tr>
<tr>
<td>5</td>
<td>G-RT-3</td>
<td>NSM FRP Bond</td>
<td>Grout</td>
</tr>
<tr>
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<td>Freeze-Thaw</td>
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</tr>
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<td>Freeze-Thaw</td>
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</tr>
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</tr>
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</tr>
<tr>
<td>21</td>
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Table 3.5: Direct pull-out testing program

<table>
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Figure 3.1: Slab strip loading frame and test setup

Figure 3.2: Elevation (a) and section (b) views of the slab strip dimensions and reinforcement details (after Burke, 2008)
Figure 3.3: Slab strip formwork with steel reinforcement in place prior to casting the concrete (after Burke, 2008)

Figure 3.4: Consolidating the fresh concrete in the formwork
Figure 3.5: Tuckpoint grinder used to cut grooves for NSM in the concrete cover

Figure 3.6: Two views of the aluminum angle grinder guide and slab strip prior to cutting an NSM groove (after Burke, 2008)
Figure 3.7: Cutting NSM groove in slab strip specimen using aluminum jig (after Burke, 2008)

Figure 3.8: 100mm gauge length Pi Gauge used in the current study

Figure 3.9: Caulking and foam barriers used to dam the polymer or grout adhesive in the NSM groove during strengthening installation (after Burke, 2008)
Figure 3.10: Thermocouples installed on a thermocouple frame within in slab-strip formwork prior to casting the concrete
Figure 3.11: Unprotected strain gauge (top) and strain gauge protected with marine grade black silicone (bottom) coupled with wiring in sealed black tubing to prevent damage to the strain gauge due to exposure to water in freeze-thaw tank.
Figure 3.12: Schematic showing location of loading points and locations of Pi gauges (PI), linear potentiometers (LP) and foil strain gauge (SG) (after Burke, 2008)

Figure 3.13: Freeze-thaw temperature cycles for a typical slab strip specimen during a 24 hour period
Figure 3.14: Sustained load being applied to a pair of back-to-back slab strip specimens using a hydraulic jack and two load cells

Figure 3.15: Sustained loading setup after load is applied to a pair of back-to-back slab strip specimens
Figure 3.16: Freezing in air (left) and thawing in water (right) of slab strip specimens under sustained load.

Figure 3.17: The C-shaped concrete block designed by De Lorenzis et al (2002) that was used for the modified pull-out testing program.
Figure 3.18: Plan (a) and elevation (b) views of the direct pull-out specimens used in the current study

Figure 3.19: Formwork used for the modified pull-out specimens
Figure 3.20: 3-D computer representation and actual pull-out specimen in the test setup in preparation for testing. The aluminum cylinder was only used to support the grips prior to testing and was no longer needed once the hydraulics were running.
Figure 3.21: High resolution digital image taken for pixel tracking of displacements and optical strain measurement during pull-out testing
Chapter 4

Results and Discussion

4.1 Introduction

The results of both the slab strip and modified pull-out testing program experiments are presented and discussed in this chapter. In addition, the results of ancillary tests – some of which were performed by others (as noted) – on the concrete, reinforcing steel, and FRP are also presented. Both the slab strip and pull-out testing programs are discussed with respect to the conditioning regimes they experienced prior to testing (i.e., exposure to freeze-thaw cycles, sustained load, sustained load and freeze-thaw cycles, etc.). In Chapter 5 the experimental results are compared against predictions from a numerical layer model and against the predictions of an existing FRP-to-concrete bond model developed previously by Seracino et al. (2007a).

4.2 Ancillary Test Results

To predict the response of the slab strips using the numerical layer model described in Chapter 5, ancillary tests were performed on the slab strips’ constituent materials to determine the requisite model input parameters for material strength, stiffness, and stress-strain response. The results of these tests are summarized in Tables 4.1 through 4.4 and are discussed briefly in the following sections.

4.2.1 Concrete

Results from all of the tests performed on standard 150 mm diameter by 300 mm tall concrete cylinders are summarized in Table 4.1. The concrete had a specified 28 day compressive strength of 35 MPa. Testing revealed that the actual 28 day strength was 44 MPa
(average) ± 2.4 MPa (one standard deviation). Additional identical concrete cylinders were also tested at the time of slab strip testing (517 days), as shown in Table 4.1. A set of four cylinders was left at room temperature in the Structural Engineering Laboratory while the slab strips underwent exposure to their respective preconditioning regimes, and another set of four cylinders was exposed to 300 freeze-thaw cycles along with the freeze-thaw slab strips. The choice of four cylinders rather than three was essentially arbitrary but provides slightly more confidence in the results obtained. Testing of these cylinders at 517 days produced strengths of 41 MPa ± 2.9 MPa and 50 MPa ± 8.6 MPa for the room temperature and freeze-thaw cycled cylinders, respectively. The slight apparent increase in strength of the cylinders which were subjected to freeze-thaw cycles may be due to the additional wet curing time provided by the water used to thaw the concrete after each freezing cycle, which may have more effectively cured the concrete. Nonetheless, these tests appear to show that the concrete itself was not damaged by exposure to 300 freeze-thaw cycles (as expected given that it was air entrained, as discussed in Chapter 3). Although the cylinders tested at 517 days after remaining at room temperature produced a slightly lower average strength (by 8%) compared to the cylinders tested at 28 days, the difference is not large and the standard deviations of the data sets suggest that the samples may have had statistically similar compressive strengths.

4.2.2 CFRP

The properties of the carbon FRP strips used in the testing program were determined previously by Burke (2008) through a series of tensile coupon tests. Burke (2008) performed research to study the response at low and high temperature of NSM FRP strengthening systems that were essentially identical to those tested in the current study. Burke’s (2008) coupon test results are summarized in Table 4.2. Tests on six individual tensile coupons produced an average
rupture strength of 2780 MPa (average) ± 139 MPa (one standard deviation) with a corresponding ultimate strain of 2.0%. The modulus of elasticity, determined to be 141 GPa ± 4.0 GPa, was calculated as the slope of the recorded stress-strain data between 1000 µε and 3000 µε. These data agree reasonably well with the manufacturer’s data, also given in Table 4.2, particularly considering that the manufacturer’s data represent mean values less three standard deviations.

4.2.3 Reinforcing Steel

The mechanical properties of the D5 reinforcing wire which was used as longitudinal internal reinforcing steel in the tests presented in this thesis were determined by Ranger (2007), using steel wire from the same coil as used herein, through a series of direct tensile tests. The results of these tests are summarized in Table 4.3. The steel displayed a 0.2% offset yield strength of 667 MPa ± 12.0 MPa, with a corresponding yield strain of 0.55% ± 0.015%. The elastic modulus of the steel was calculated as 194 GPa ± 15.7 GPa. Note that this steel was cold-drawn deformed wire which had been straightened after being supplied in a coil. As a result, the material did not display a well defined yield point, and the material properties were defined using a 0.2% offset yield definition.

4.2.4 Mortar Cubes (Grout Testing)

Burke (2008) constructed and tested a series of mortar cubes using the Target 1118 cementitious grout adhesive that was used in the current testing program to determine the compressive strength of the cementitious adhesive and to ensure that the water-to-grout ratio used during mixing resulted in satisfactory hardened properties. Testing of three grout mortar cubes produced a compressive strength of 61.3 MPa ± 5.5 MPa. Results from these tests are given in Table 4.4. The same water-to-grout ratio as verified by Burke (2008) was used when applying the grout adhesive NSM strengthening used in the current study.
4.3 Slab Strip Testing Program

As previously stated, a total of 21 slab strips were tested for the purposes of the current thesis. The tests were performed after the slab strips were exposed to one of four conditioning regimes. These were:

1. room temperature (RT);
2. sustained load (SL);
3. freeze-thaw cycling (FT); or
4. freeze-thaw cycling under sustained load (FT-SL).

The results obtained during the testing of the specimens are presented and discussed in the following sections. A selection of results from the slab strip testing program is given in Table 4.5. These results are also compared graphically on the basis of ultimate total applied load (Figure 4.1), and peak strain in the FRP as a fraction of ultimate strain (Figure 4.2). The applied load versus midspan deflection, midspan moment versus midspan curvature, and midspan moment versus midspan FRP strain for all epoxy and grout adhesive strengthened slab strips are summarized in Figure 4.3 through Figure 4.8 to provide clear contrast of the slab strip performances after different conditioning regimes. For the purposes of comparison, two tests of essentially identical unstrengthened slab strips and two tests of essentially identical epoxy strengthened slab strips taken from the previous study by Burke (2008) are included in the following results. When included, Burke’s data are shown in tables using italic font, and in figures using dashed lines.

4.3.1 Room Temperature Slab Strips
A set of five slab strips were left at room temperature after strengthening to act as unexposed control specimens. Of these five slab strips, two were strengthened with a single carbon FRP NSM strip using the epoxy adhesive and three were strengthened using the grout. The experimental program was originally planned such that three slab strips using each adhesive type would be exposed to each conditioning regime. However, as previously mentioned two of the epoxy adhesive strengthened slab strips did not cure properly, and one of the slab strips was irreparably damaged prior to strengthening. For those reasons, three of the conditions involving epoxy adhesive strengthening (room temperature, freeze-thaw exposure, and freeze-thaw cycling while under sustained load) had only two test specimens per treatment.

The total applied load versus midspan deflection response of the room temperature slab strips during testing is shown in Figure 4.9. The load-deflection behaviour was characterized by a large initial stiffness before cracking of the concrete in tension at the extreme tension fibre, followed by a gradual reduction in stiffness with subsequent drops and increases in load corresponding to the formation of additional cracks in the cross-section. The results from two unstrengthened reinforced concrete slab strips (C 1 and C 2), which have been taken from Burke (2008), are included for reference and to demonstrate the degree of strengthening that was achieved. Burke’s slab strips were cast from a different concrete batch, but the specified mix design was identical and the compressive strength testing discussed previously produced similar values to the concrete used in the testing program presented in this thesis (Burke’s concrete had a 28 day compressive strength of 42 MPa, as compared with 44 MPa in the current study). The results from E RT 1 are not included because of a data acquisition malfunction caused by a faulty ground. To provide additional comparison however, two similar room temperature slab strips strengthened with epoxy adhesive from Burke (2008) are included in the figure. Furthermore,
linear potentiometers (LP) malfunctioned during testing of G RT 1 and prevented the generation of a load-deflection plot for this specimen.

As expected, all strengthened slab strips showed greater stiffness than unstrengthened specimens after cracking. The epoxy strengthened slab strips (including Burke’s (2008) data) achieved ultimate loads of between 32.6 kN and 34.9 kN, while the grout strengthened slab strips reached loads between 27.2 kN and 28.4 kN. The strengthened slab strips achieved much higher ultimate loads (by 129% to 190%) than the unstrengthened reinforced concrete slab strips tested by Burke (2008), which reached loads of 11.4 kN and 12.7 kN respectively. Aside from noticeable increases in ultimate load, the strengthened slab strips also provided improved flexural deformability (in terms of deflection). All strengthened slab strips deflected by at least an additional 50% compared with the unstrengthened specimens. The midspan moment versus midspan curvature relationships for the room temperature slab strips (in which curvatures are derived from Pi Gauge data collected during testing) are shown in Figure 4.10. The unstrengthened members displayed similar behaviour prior to cracking. After cracking, however, the presence of NSM FRP drastically increased the stiffness of the strengthened cross-sections. Although one of the unstrengthened slab strips failed with less curvature than the strengthened slab strips, one of the often-cited advantages of traditional steel reinforcement is the capability of the beams to develop large curvatures at failure (in excess of 0.00035 for C 1, for instance) providing ample warning of failure. Although the FRP debonding occurred suddenly and essentially without warning, the slabs developed reasonably large curvatures (almost all in excess of 0.00015) which would have likely provided ample warning of a serious structural issue in a real strengthening application with similar proportions. The ACI Guide for the Design and Construction of Externally Bonded FRP systems for Strengthening Concrete Structures (ACI,
includes ductility guidelines to ensure proper warning of failure. The document highlights that for “reinforced concrete members with non prestressed steel reinforcement, adequate ductility is achieved if the strain in the steel at the point of concrete crushing or failure of the FRP, including delamination or debonding, is at least 0.005”. For the case of these slab strips, the lowest FRP strain at failure was 0.008, which corresponds to a steel strain in excess of 0.006 (by assuming a conservative neutral axis depth of 15 mm and a linear strain profile). According to ACI 440 guidelines (ACI, 2008), adequate ductility was achieved.

Consideration of the midspan moment versus midspan FRP strain relationship (shown in Figure 4.11) highlights the similarities and differences in the behaviour of the epoxy adhesive and grout adhesive strengthened members prior to failure, which was by sudden FRP debonding in all cases. All three of the grout adhesive strengthened slab strips failed between 6.9 kN-m and 7.2 kN-m with FRP strains at failure between 0.97% to 0.99%. Considering the manufacturer quoted ultimate strain in the FRP was 1.7%, the grout adhesive strengthened slab strips averaged 58% of ultimate FRP strain at failure, less than the 70% maximum strain capacity currently allowed by ACI 440.2R-08 in NSM strengthening applications using epoxy adhesives (ACI, 2008). The epoxy adhesive strengthened slab strips exceeded both the ultimate loads and FRP strains of the grout adhesive strengthened slab strips, with failure occurring between 8.3 kN-m and 8.9 kN-m with corresponding FRP strains in the range of 1.26% to 1.41%. The average FRP strain of 78% of ultimate was in excess of the current ACI 440.2R-08 guidelines (ACI, 2008).

4.3.2 Slab Strips Exposed to 300 Freeze-Thaw Cycles

A set of five slab strips was exposed to 300 freeze-thaw cycles. Of the five specimens, two were strengthened using epoxy adhesive, and three were strengthened using cementitious grout. As previously mentioned, this is because the experimental program was reduced by three
epoxy slab strips during the fabrication stages of the project. The applied load versus midspan deflection performance of the freeze-thaw slab strips is shown in Figure 4.12. The epoxy adhesive strengthened slab strips achieved ultimate loads of 33.1 kN and 32.6 kN, respectively, while the grout strengthened slab strips reached loads between 26.6 kN and 29.5 kN. The specimens all displayed similar stiffness up to load levels approaching ultimate, with some variability of stiffness observed in the grout strengthened members. Compared to the room temperature specimens discussed previously, the slab strips exposed to 300 freeze-thaw cycles did not experience drops in load after the appearance of the first crack in the tensile region of the concrete. Additionally, the load-deflection response was much smoother, with fewer drops in load, likely due to a finer and more gradual progression of cracking during loading. One possible explanation for this behaviour is that the freeze-thaw cycling induced a network of micro-cracks into the concrete through the freezing and thawing of pore water. Then, as the slabs were loaded, the large scale tension cracks could have formed by gradually progressing through pre-existing microcracks, previously initiated by the freeze-thaw cycles. This gradual crack formation would have developed without the sudden release of energy (and subsequent drop in load) that was typical with the room temperature conditioned slab strips.

The moment versus midspan curvature relationships for the freeze-thaw cycled specimens (Figure 4.13) shows an abnormality for E FT 1, which displayed a sudden jump and less curvature than expected beyond an applied moment of about 3.2 kN·m. This may be the result of a rigid movement of one of the Pi Gauges, although the true source remains a mystery. The author is unable to provide any credible physical reason for this strange moment-curvature response. Data are not presented for the moment-curvature response of E FT 2; this could not be determined because the Pi Gauges fell off the side of the slab strip during testing and all Pi Gauge
data were lost. The midspan moment versus midspan FRP strain response for these specimens, shown in Figure 4.14, shows the overall high similarity in the behaviour of the slab strips subjected to freeze-thaw cycling. These data also confirm the hypothesis that the strange moment-curvature response of specimen E FT 2 noted above was due to an instrumentation issue rather than a physical reality, since no sudden changes in FRP strain are observed at the same load level. It should be noted, however, that the epoxy adhesive strengthened specimens failed at larger moments (8.3 kN·m and 8.4 kN·m) and FRP strains (between 1.1% and 1.2%) compared to the grout adhesive strengthened slab strips (which failed at moments ranging from 7.0 kN·m to 7.5 kN·m, with corresponding FRP strains of about 0.9%). After freeze-thaw cycling, both the epoxy adhesive and grout adhesive strengthened slab strips failed to attain the maximum level of FRP strain permitted by ACI 440.2R-08 (ACI, 2008). The epoxy adhesive strengthened strips averaged 69% of the ultimate strain of the FRP at failure. The grout adhesive strengthened slab strips used 15% less of the full potential of the FRP, averaging only 54% of ultimate FRP strain capacity. Although lower FRP strains at failure might discourage the reader as to the potential of NSM FRP in freeze-thaw environments, when examining the similar (and in the case of the grout adhesive, increased) ultimate loads after 300 freeze-thaw cycles, NSM FRP certainly shows promise for exterior applications in cold climates.

4.3.3 Slab Strips Exposed to Sustained Load at Room Temperature

A set of six slab strips was exposed to a period of sustained load at room temperature. Of the six specimens, three were strengthened using epoxy adhesive, and three were strengthened using cementitious grout. One of the epoxy adhesive strengthened slab strips failed during handling while it was being loaded into the test setup, and thus no data are available for this specimen. This was because the cross-section was pre-cracked due to the relatively large
sustained load which had been applied. As mentioned in Chapter 3, the test was setup such that the bond line was on the top side of the slab strip to provide visual access to the bond during testing. The slabs were thus tested upside down. This proved to be problematic in terms of loading the sustained load conditioned slabs into the test frame. When the first test specimen was flipped over to be placed under the loading beam, the small portion of the cross-section which was uncracked was placed in tension and it cracked and the slab strip folded into two pieces connected by nothing but a buckled strip of NSM FRP. To circumvent this problem for subsequent tests, the remainder of the slab strips which had been exposed to sustained load were placed into the loading frame using a three meter long piece of timber. Once the slab strip was carefully rolled over so the FRP strip was facing up, the timber was placed on top of the slab strip. Then, using ratchet tie straps, the slab strip was attached to the timber at the same points where the sustained load had been applied. Once tied together, the timber could be lifted up into place, and the slab strip was supported from underneath at two points while being loaded into the testing frame.

The load versus midspan deflection response of the sustained loaded slab strips during testing is shown in Figure 4.15. The epoxy adhesive strengthened slab strips achieved ultimate loads of 32.8 kN and 33.3 kN, while the grout strengthened slab strips reached loads between 29.9 kN and 31.6 kN. Specimen G SL 2 showed slightly more deflection for a given load compared to the other slab strips. The moment versus midspan curvature relationship (shown in Figure 4.16) again shows an abnormality for G SL 2, with much more curvature than expected during the initial 4 kN·m of increasing moment, followed by relative increases in moment which better match the other four specimens when the applied moment was increased beyond 4 kN·m. This behaviour may have been the result of a flexural crack which developed within the gauge
length of the Pi Gauges during the period of sustained load. Given the geometry used in the sustained load setup, 15 kN of load corresponds to a moment of 3.81 kN·m. A flexural crack would therefore have gradually re-opened with less applied moment than a section which was previously uncracked.

The moment versus FRP strain response, shown in Figure 4.17, suggests the behaviour of all 5 strips was similar, with the exception of the epoxy strengthened slab strips reaching greater ultimate moments (8.3 kN·m and 8.4 kN·m) and associated FRP strains (1.2% to 1.3%) while the grout strengthened beams failed at very slightly lower applied moments (7.6 kN·m, 7.6 kN·m, and 8.0 kN·m) and associated FRP strains (1.0% to 1.1%). The epoxy adhesive strengthened slab strips generated strains in excess of the ACI 440.2 limit, at 73% of ultimate FRP strain, and the grout adhesive strengthened slab strips fell only 8% short of 70% on average, failing at an average of 62% of the ultimate FRP strain. The sustained load slab strips proved to be situated with the most similar failure strains between the epoxy and grout adhesives. It would not be unreasonable to assume that the polymer resin of the epoxy was influenced by the sustained load and experienced some level of creep. At the same time, it is possible that the same mechanism which produces increases in concrete strength under sustained load (described previously in Chapter 2) may have also produced an increase in grout strength under sustained load.

### 4.3.4 Slab Strips Exposed to 300 Freeze-Thaw Cycles while Under Sustained Load

A set of five slab strips was exposed to 300 freeze-thaw cycles while under a sustained load of 15 kN. Of the five, two were strengthened using epoxy adhesive, and the remaining three were strengthened using grout adhesive. Results of the applied load versus midspan deflection relationships obtained during the testing of these slab strips are shown in Figure 4.18. Again, the
specimens strengthened with epoxy adhesive attained the greatest ultimate loads and showed the
greatest deflections before failure. The epoxy adhesive strengthened slab strips reached ultimate
loads of 32.0 kN and 30.0 kN, whereas the grout adhesive strengthened slab strips reached
slightly lower ultimate loads between 27.6 kN and 29.3 kN.

The midspan moment versus midspan curvature of the freeze-thaw and sustained load
conditioned slab strips is displayed in Figure 4.19. Similar to one of the grout adhesive
strengthened slab strips from the sustained load conditioning regime discussed above, all of the
slab strips in this scenario experienced much more curvature than expected during the first 4
kN·m of moment. Beyond 4 kN·m, however, the slope of the midspan moment versus midspan
curvature response is similar to all other specimens. As for the isolated case described in the
previous section, it is believed that a flexural crack opened up at midspan while the sustained load
was applied to these specimens. During testing, this crack would have provided an area of high
localized curvatures. Once the applied load exceeded the load previously applied to slab strips
the relative increases in curvature for each increase in applied moment match the other tests. This
behaviour can be verified by examining the moment versus midspan curvature for all grout slab
strips and all epoxy slab strips in Figure 4.4 and Figure 4.7, respectively.

The midspan moment versus midspan FRP strain relationships for these beams are
presented in Figure 4.20. The slopes of these curves are essentially constant until the internal
reinforcing steel yielded because the slab strips were already cracked prior to testing. The
sustained load of 15 kN was well above the cracking load for the strengthened specimens (the
cracking load has been shown in other load versus deflection diagrams discussed previously and
was consistently less than 5 kN). Neither the epoxy adhesive nor the grout adhesive slab strips
produced strains in excess of the current ACI 440.2R-08 (ACI, 2008) design strain limit for NSM
FRP. The epoxy and grout adhesive strengthened slab strips exposed to freeze-thaw cycles and sustained load reached average maximum FRP strains of 63% and 53% of the ultimate strain of the FRP, respectively. Although the strains for both sets of specimens dropped compared to the controls, when the loads are compared it can be seen that the grout adhesive slab strips actually produced improved ultimate load capacity after freeze-thaw cycling under sustained load. The epoxy, on the other hand, displayed a drop in ultimate load as well as a drop in ultimate strain, however this reduction was relatively small (approximately 8%). From considering these data, it can be concluded on a preliminary basis that 300 freeze-thaw cycles during sustained load does not produce drastic changes in behaviour or reductions in ultimate capacity for either type of adhesive.

4.4 Modified Pull-Out Results

As previously stated, a total of 16 modified pull-out bond test specimens were tested for the purposes of the current thesis. These tests were performed after the slab strips were exposed to one of two conditioning regimes:

(1) room temperature (RT); or

(2) freeze-thaw cycling (FT).

The results obtained during the testing of the specimens are presented and discussed in the following sections. The overall results from the modified pull-out testing program are summarized in Table 4.6 and Table 4.7. In the case of the grout adhesive, bond failure was always located at the interface between the FRP strip and the grout adhesive, so that failure was due to the FRP tape slipping out of the adhesive at the loaded end of the bond. A photo of what appears to be intact grout after what is in fact a bond failure is shown in Figure 4.28. In the case
of the epoxy adhesive, the failures were located in the concrete adjacent to the epoxy adhesive, and in some cases caused failures of a substantial portion of the pull-out blocks. Depending on the failure load, some of the pull-outs strengthened with epoxy had a large wedge of concrete dislocate upwards from the specimen along with the NSM strip. One such failure is shown in Figure 4.29. In general, the epoxy adhesive bonded lengths outperformed the grout adhesive bonded lengths after exposure to both room temperature or 150 freeze-thaw cycles. However, after exposure to 150 freeze-thaw cycles, the pull-out load of the epoxy specimens was reduced by 33% on average as compared with the room temperature specimens, whereas the grout adhesive produced slightly greater ultimate pull-out loads (on average) after freeze-thaw cycling, likely because of additional curing of the grout due to the water used in the thawing phases of the freeze-thaw cycles. Specifics of the test data are discussed in the following sections.

4.4.1 Room Temperature Pull-Out Results

Of the eight pull-out tests which were maintained at room temperature conditions to act as control specimens, four were strengthened using epoxy adhesive and the remaining four used grout adhesive. The average pull-out load for the epoxy specimens was 53.5 kN ± 2.5 kN, with a corresponding average strain in the FRP of 1.20% ± 0.05%. The epoxy pull-outs therefore reached the ACI 440.2R-08 (ACI, 2008) FRP strain limit, failing slightly above 70% of the ultimate FRP strain. The average load of the grout adhesive bonded lengths was considerably lower, at an average of 24.9 kN ± 6.8 kN. The grout adhesive also produced considerably lower peak strains, with the four specimens averaging 0.53 % ± 0.11%. Considering that the manufacturer guarantees an FRP failure strain of 1.7%, 0.53% strain at failure corresponds to only 31% of the FRP’s ultimate strain capacity, which represents relatively poor material efficiency. Graphs showing each test result as well as the average for each group are given in
Figure 4.21. The failure strain as a fraction of the ultimate FRP strain is displayed in Figure 4.22. A plot comparing the pull-out load versus the stroke of the hydraulic actuator used to apply the pullout loads during the test (with the elastic strain from the free length of FRP removed) is shown in Figure 4.23. Note that the initial stiffening of the load versus stroke plot is likely due to seating of the wedge action grips during testing, and it is thus not representative of any true mechanical stiffening of the bonded lengths. Unfortunately it was not possible to directly measure loaded end slip for the epoxy adhesive pullout specimens due to cracking of the concrete and wedge formation at the loaded end of the bond as described previously. The loaded end slip response of the grout specimens was approximated using digital image analysis and is discussed below. The general behaviour of the epoxy and grout specimens followed the same trend, with the exception that the grout adhesive pull-out specimens failed at much lower loads (24.9 kN on average) as compared to the epoxy pull-outs (53.5 kN on average).

4.4.2 Pull-Out Specimens Exposed to 150 Freeze-Thaw Cycles

Of the eight pull-out tests which were exposed to 150 freeze-thaw cycles, four were strengthened using epoxy adhesive, and the remaining four used grout adhesive. The average pull-out load for the epoxy adhesive bonded specimens was 38.9 kN ± 4.1 kN, with a corresponding average strain in the FRP of 0.91% ± 0.10%. This load corresponds to an average 27% reduction in ultimate load as compared to the room temperature bond pull-out tests using epoxy adhesive. It is believed that the difference in coefficients of thermal expansion between the concrete, the FRP, and the epoxy adhesive played the central role in this observed bond strength degradation, since differential thermal expansion would generate stresses in the bond line, potentially causing microcracking within the concrete adjacent to the epoxy. This result
suggests that the bond strength of NSM strips using an epoxy adhesive may indeed be adversely affected by repeated freeze-thaw cycling.

The average ultimate load of the pull-out specimens which used grout adhesive was 28.1 kN ± 7.0 kN; slightly higher than the value produced for the room temperature exposure. Similar to the concrete cylinders which were freeze-thaw cycled, the grout adhesive was likely stronger after freeze-thaw cycling due to the additional wet curing time provided during the thawing phase of each freeze-thaw cycle. The grout adhesive strengthened pull-out specimens also displayed larger peak FRP strains after freeze-thaw cycling, with the four specimens averaging 0.65% ± 0.16%. Graphs showing each test result, as well as the average for each group are displayed in Figure 4.13 and Figure 4.14 for both ultimate pull-out load and ultimate FRP strain, respectively. It is also likely that the cementitious grout had a similar coefficient of thermal expansion to the substrate concrete, so that differential thermal expansion between the adhesive and the surrounding concrete would be less likely, thus reducing the likelihood of bond damage during cycling.

Plots showing the pull-out load versus the stroke of the hydraulic actuator used to apply the pullout loads (less the elongation of the free length of FRP) for the grout and epoxy specimens after being subjected to 150 freeze-thaw cycles are shown in Figure 4.24 and Figure 4.25, respectively. Both graphs show that the general trend of the bond test behaviour after exposure to 150 freeze-thaw cycles does not change, with the exception of the failure load being reduced for the epoxy adhesive strengthened specimens. Some curves appear to be smooth lines, and others appear more ‘choppy’ in nature. It is believed that the curves with the continual ridges represent small slips in the gradual seating of the wedge action grips of the actuator. Why this behaviour was not observed for all tests, however, cannot be explained.
4.5 Effect of Adhesive Type

4.5.1 Slab Strips

As previous research has suggested should be the case (Burke, 2008), the epoxy adhesive outperformed the grout adhesive for the unexposed slab strips. Unfortunately, only one of the epoxy strengthened slab strips was able to produce meaningful data due to issues with the data acquisition system. This single epoxy slab strip achieved 117% of the average grout adhesive strengthened slab strip ultimate load. When combined with data from the two essentially identical epoxy adhesive strengthened slab strips tested by Burke (2008), the epoxy adhesive strengthened slab strips averaged an ultimate load of 33.7 kN, greater than the 27.9 kN average load of the grout adhesive strengthened slab strips. It is believed that the greater tensile strength of the epoxy prevented slip failure at the adhesive/FRP interface, and eventually forced the failure into the concrete adjacent to the epoxy when the tensile strength of the concrete was reached. The effect of this would be to increase the shear perimeter and area of the failure plane and thus to increase the bond strength (assuming similar tensile strengths for the concrete and the grout). Images of a typical grout slab strip and epoxy slab strip after failure are shown in Figure 4.26 and Figure 4.27, respectively.

The ultimate loads and FRP strains normalized against the ultimate strain of the FRP are displayed in Figure 4.1 and Figure 4.2. Results from two tests on epoxy strengthened slab strips by Burke (2008) are included to complement that data from the current research program. The bar graphs which compare peak strains also display the currently imposed ACI 440.2R-08 (ACI, 2008) strain limit. Epoxy adhesive strengthened specimens tested after room temperature and sustained load conditioning regimes both exceeded the current ACI 440 strain limit. After 300 freeze-thaw cycles, the epoxy slab strips gave FRP strains slightly below the current ACI 440
strain limit, failing at an average FRP strain of 69% of the ultimate FRP strain. After 300 freeze-
thaw cycles while under sustained load, the epoxy slab strips failed at an average FRP strain of
63% of ultimate, whereas the grout slab strips failed with average FRP strains ranging from 53%
to 62%. The grout slab strip regime which most effectively used the FRP’s ultimate strain
capacity was the room temperature sustained load condition (using 62% of the FRP’s ultimate
strain).

Similar to the epoxy adhesive strengthened slab strips, the lowest FRP strains in the case
of grout adhesive were produced after freeze-thaw cycling under sustained load, where the grout
adhesive strengthened slab strips failed at an average of 53% of ultimate FRP strain. The room
temperature specimens failed at an average of 58% of ultimate strain in the FRP, while the freeze-
thaw conditioned specimen used 4% less of the ultimate strain in the FRP, failing an average
level of 54%.

4.5.2 Modified Pull-Out Bond Specimens

The modified-pull out tests displayed superior performance for the epoxy adhesive in all
tests. The four epoxy pull-out specimens tested at room temperature averaged more than double
(215%) of the ultimate load of the four grout specimens under the same conditioning. A visual
comparison of ultimate loads and FRP strains achieved for the pull-out bond tests is displayed in
Figure 4.21 through Figure 4.22. The epoxy adhesive specimens used 70% of the ultimate strain
of the FRP, thus reaching the current ACI 440.2R-08 (ACI, 2008) FRP strain limit. The grout
adhesive strengthened specimens averaged less than half of that the epoxy adhesive value, using
only 31% of the ultimate strain capacity of the FRP material. It is believed that the lower tensile
strength of the grout as compared to the epoxy allowed the failure to shift from the concrete
inwards to the interface between the FRP and the grout adhesive.
For the case of a shear strengthened member, the ACI 440.2R-08 guidelines (ACI, 2008) recommend limiting the strain in the FRP to 0.4% in order to maintain aggregate interlock. The epoxy adhesive averaged three times this value (1.20%). While the grout adhesive produced lower failure strains, every test using grout adhesive failed at an FRP strain above 0.4, with the data set averaging 0.53% strain. Therefore, although the grout adhesive was not sufficiently strong to develop the maximum strains recommended for flexural strengthening design, the grout adhesive did develop the necessary strains to exceed the maximum recommended strains allowable for shear strengthening design. Note that the bond pull-out tests are much more similar to a shear strengthening application than a flexural one, providing further confidence in the ability to use cementitious grout adhesives in shear strengthening applications for concrete structures.

In general, on the basis of the test data presented in this thesis it appears that both epoxy adhesives and cementitious grouts can be used to apply NSM FRP strengthening systems to reinforced concrete structural members, but that more stringent strain limits, probably in the range of 0.4% should be applied when using grout adhesives. For epoxy adhesives the currently suggested strain limit of 70% of the tensile strain capacity of the FRP appears to be reasonable in the absence of any deterioration of the FRP system due to freeze-thaw cycling.

4.6 Effect of Freeze-Thaw Cycling

4.6.1 Slab Strips

All slab strips tested after freeze-thaw cycling without sustained load produced similar results to the room temperature slab strips, with only very minor reductions in strength or strain in the FRP at failure due to freeze-thaw cycling. The epoxy adhesive strengthened slab strips failed at an average load of 33.7 kN without freeze-thaw cycling, and an average of 32.8 kN after 300
freeze-thaw cycles. The grout slab strips failed at an average of 27.9 kN after room temperature exposure, and 28.3 kN after 300 freeze thaw cycles. The FRP strain at failure, was slightly reduced after exposure to freeze-thaw cycles. The epoxy specimens displayed an average FRP failure strain drop from 1.31% to 1.17% and the grout specimens experienced a similar drop from 0.98% to 0.91%. Also, as expected, there was a marginal increase in stiffness with freeze-thaw cycling (shown as a marginal increase on the slope of load versus deflection graph in Figure 4.3 and Figure 4.6) likely due to the increase in compressive strength of the concrete due to the additional wet curing provided during freeze-thaw cycling. After 300 freeze-thaw cycles both the epoxy and the grout adhesive strengthened slab strips failed to reach the currently imposed ACI 440.2R-08 strain limit for flexural strengthening (ACI, 2008). However, despite minor reductions in strain, 300 freeze-thaw cycles does not appear to drastically impact slab strips strengthened in flexure with NSM FRP.

### 4.6.2 Modified Pull-Out Bond Specimens

The epoxy adhesive pull-out specimens displayed a 27% decrease in average ultimate load after freeze-thaw cycling. The grout adhesive pull-out specimens, on the other hand, displayed a 13% increase in average ultimate load after freeze-thaw cycling. As previously noted, the improved grout performance is likely due to the improved grout strength due to the additional wet curing time provided by the freeze-thaw cycles. It should be noted, however, that even with a decrease in load after environmental exposure, the epoxy adhesive specimens still displayed greater ultimate strength than the grout adhesive specimens. The epoxy adhesive pull-out specimens averaged an ultimate load of 38.9 kN after cycling, compared to 28.1 kN for the grout adhesive specimens. The average strain reached in the epoxy adhesive pull-out tests after 150 freeze that cycles was 0.91%, or 54% of the ultimate strain of the FRP. The average strain
reached in the grout adhesive pull-outs after 150 freeze-thaw cycles was 0.65%, or 38% of the ultimate FRP strain. From a shear strengthening perspective, both adhesives were sufficiently strong to allow the FRP strain to exceed the 0.4% limit recommended for FRP shear reinforcement outlined by ACI 440.2 (ACI, 2008). Although these data suggest that 150 freeze-thaw cycles does not limit the performance of NSM FRP for shear strengthening scenarios based on current ACI 440.2R-08 guidelines (ACI, 2008), it remains unknown if the apparent bond degradation would have been more severe had the pull-out specimens been exposed to 300 freeze-thaw cycles.

**4.7 Effect of Sustained Load**

The slab strips which were exposed to a period of sustained load at room temperature prior to testing displayed the greatest average ultimate load of any group. The average ultimate load of the epoxy adhesive strengthened slab strips decreased slightly from 33.7 kN to 33.1 kN after the period of sustained load, but the average ultimate load of the grout adhesive strengthened slab strips increased from 27.9 kN to 30.5 kN. Similar to the freeze-thaw specimens, the increase was likely due to an increase in the strength of the concrete (and likely the grout also). This hypothesis stems from the fact that although the ultimate loads of the two epoxy strengthened slab strips were within two percent of the room temperature controls, the strains in the FRP at failure for the sustained load specimens averaged six percent lower than the value observed in the room temperature tests. The average peak strain for the epoxy adhesive strengthened specimens dropped from 1.31% to 1.23%. After the sustained load exposure, the epoxy adhesive strengthened slab strips still exceeded the maximum recommended FRP strain as imposed by ACI 440.2R-08 (ACI, 2008). The average peak strain for the grout adhesive strengthened specimens increased from 0.98% (in the room temperature control tests) to 1.06% after 120 days of
sustained load. Even with the increase in peak FRP strain, the FRP strains observed in the grout adhesive strengthened slab strips remained below the maximum recommended FRP strain for flexural strengthening outlined by ACI 440.2R.08 (ACI, 2008). Despite the somewhat limited number of specimens tested for this thesis, sustained load does not appear to be a problem for either of these adhesive systems.

4.8 Effect of Combined Freeze-Thaw Cycling and Sustained Load

The two adhesive systems displayed different behaviour after exposure to a period of sustained load during freeze-thaw cycling. The epoxy adhesive strengthened slabs displayed a slight reduction in both average ultimate load (from 33.7 kN to 31.0 kN) and average FRP strain at failure (from 1.33% to 1.08%). The grout slabs also displayed a slight reduction in FRP strain at failure (from 0.98% to 0.89%). The ultimate loads of the grout slabs, however, averaged higher than the room temperature slabs (28.5 kN compared to 27.9 kN). On the basis of the relatively few tests performed on slab strips exposed to 300 freeze-thaw cycles under sustained load, it would appear that the cementitious grout is not negatively influenced in terms of overall performance after this exposure condition. The increase in failure load could be attributed to greater concrete and grout strengths due to the additional wet curing in the freeze-thaw tanks. The effect of freeze-thaw cycling and sustained load on the epoxy slab strips seems to be the minimal, but in line with the slight reduction in debonding load after freeze-thaw cycling and the slight reduction in debonding load after a period of sustained load. More research is required to more appropriately examine this behaviour.
4.9 Summary

This chapter included the detailed results of a testing program consisting of tests on 21 NSM FRP strengthened reinforced concrete slab strips and 16 bond pull-out tests on NSM FRP strips anchored to unreinforced concrete blocks.

For the scenario of flexural strengthening, as examined in the slab strip tests, the effects of freeze-thaw cycles, sustained load, and freeze-thaw cycles in combination with sustained load were examined for their effects on two adhesives: an epoxy (Kemko 038) and a cementitious grout (Target 1118). The strengthened slabs displayed large increases in ultimate load capacity in all cases as compared to unstrengthened slabs tested previously by Burke (2008), with increases in load ranging from 129% to 190%. The epoxy adhesive outperformed the grout adhesive under all conditioning scenarios. The conditioning from 300 freeze-thaw cycles, sustained load, and a combination of 300 freeze-thaw cycles and sustained load caused only minimal effects on ultimate load, with reductions by as much as 8% for the epoxy adhesive, and increases by as much as 9% for the grout adhesive. The epoxy adhesive strengthened slab strip specimens that were conditioned at room temperature exceeded the ACI 440.2R-08 (ACI, 2008) recommended strain limit of 70% of the FRP failure strain for flexural strengthening, as did the epoxy adhesive slab strips conditioned under sustained load. All other scenarios, both epoxy and grout, failed to generate 70% of the ultimate strain of the FRP’s strain capacity during testing.

For the shear strengthening scenario, as examined in the modified pull-out bond tests, the effects of 150 freeze-thaw cycles were examined for the same two adhesives as were used in the flexural setting. The freeze-thaw conditioning reduced the average ultimate load of the epoxy adhesive bonded pull-outs by 27%, from 58.5 kN to 38.9 kN. The same freeze-thaw conditioning increased the ultimate capacity of the grout adhesive bonded pull-outs by 15%, from 24.5 kN to
28.1 kN. Although the epoxy adhesive was impacted by 150 freeze-thaw cycles, the residual strength after the degradation was still greater than current strengthening limits. All pull-out tests produced peak FRP strains in excess of the 0.4% FRP strain limit recommended by ACI 440.2R-08 (ACI, 2008) for shear strengthening applications with bonded FRP reinforcement.
Table 4.1: Results of concrete cylinder tests

<table>
<thead>
<tr>
<th>Cylinder Age (days)</th>
<th>Exposure Condition</th>
<th>Test Number</th>
<th>$f'_c$ (MPa)</th>
<th>Average $f'_c$ (MPa)</th>
<th>Standard Deviation (MPa)</th>
</tr>
</thead>
<tbody>
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<td>8</td>
<td>Room Temperature</td>
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<td>34.5</td>
<td>36.3</td>
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<td></td>
</tr>
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<td></td>
<td>3</td>
<td>46.6</td>
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<td>40.7</td>
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</tr>
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<td>40.1</td>
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<td>4</td>
<td>58.9</td>
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</table>

* Test Number 3 at 8 days was not used as there was a capping failure during testing which resulted in premature failure

Table 4.2: Mechanical properties of Aslan 500 CFRP based on tensile tests reported by Burke (2008)

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Ultimate Strength (MPa)</th>
<th>Modulus (GPa)</th>
<th>Strain at Failure (%)</th>
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<td>1</td>
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<td>140</td>
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<td>2.1</td>
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</tr>
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<td>2838</td>
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</tr>
<tr>
<td>Standard Deviation</td>
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<td>0.1</td>
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<tr>
<td>Manufacturer Specified*</td>
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<td>124</td>
<td>1.7</td>
</tr>
</tbody>
</table>

* As quoted by Hughes Bros. Inc. (www.hughesbros.com)
Table 4.3: Mechanical properties of D5 reinforcing steel based on tension tests reported by Ranger (2007)

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Tensile Yield Strength (MPa)</th>
<th>Tensile Yield Strain (%)</th>
<th>Ultimate Tensile Strength (MPa)</th>
<th>Tensile Elastic Modulus (GPa)</th>
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<td>650</td>
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<td>Standard Deviation</td>
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* Specimen R3 was not used in the calculation of average yield strain and elastic modulus

Table 4.4: Results from mortar cube compressive strength tests on Target 1118 unsanded silica fume grout adhesive as reported by Burke (2008)

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<tr>
<th>Mortar Cube Age (days)</th>
<th>Exposure Condition</th>
<th>Test Number</th>
<th>$f'_c$ (MPa)</th>
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Table 4.5: Selected results from the slab strip testing program

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<tr>
<th>Test</th>
<th>Specimen Name</th>
<th>Failure Load (kN)</th>
<th>Failure Moment (kN-m)</th>
<th>FRP Strain at Failure (%)</th>
<th>Strength Increase (%)</th>
<th>Failure Mode</th>
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<td>E-RT-1</td>
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<td>FRP debonding (epoxy/concrete split)</td>
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<td>1.08</td>
<td>166</td>
<td>FRP debonding (epoxy/concrete split)</td>
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<td>18</td>
<td>E-FT-SL-2</td>
<td>30.0</td>
<td>7.6</td>
<td>1.08</td>
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<td>FRP debonding (epoxy/concrete split)</td>
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<td>21</td>
<td>G-FT-SL-3</td>
<td>28.8</td>
<td>7.3</td>
<td>0.92</td>
<td>139</td>
<td>FRP debonding (FRP/grout interface)</td>
</tr>
</tbody>
</table>

* A data acquisition system malfunction rendered the acquired data useless
** The specimen failed during handling while it was being loaded into the test setup

The tests listed in Italics were performed by Burke (2008) and are included for comparison purposes
### Table 4.6: Summary of maximum loads attained during pull-out testing

<table>
<thead>
<tr>
<th>Adhesive Type</th>
<th>Exposure Condition</th>
<th>Test Number</th>
<th>Specimen Name</th>
<th>Maximum Load (kN)</th>
<th>Average (kN)</th>
<th>Standard Deviation (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy</td>
<td>Control</td>
<td>1</td>
<td>E-RT-1</td>
<td>50.6</td>
<td>53.5</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>E-RT-2</td>
<td>52.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>E-RT-3</td>
<td>55.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>E-RT-4</td>
<td>55.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>150 Freeze-Thaw Cycles</td>
<td>1</td>
<td>E-FT-1</td>
<td>40.9</td>
<td>38.9</td>
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<td></td>
<td></td>
<td>2</td>
<td>E-FT-2</td>
<td>34.5</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>3</td>
<td>E-FT-3</td>
<td>43.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>E-FT-4</td>
<td>36.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grout</td>
<td>Control</td>
<td>1</td>
<td>G-RT-1</td>
<td>32.2</td>
<td>24.9</td>
<td>6.8</td>
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<td>2</td>
<td>G-RT-2</td>
<td>28.2</td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>G-RT-3</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>G-RT-4</td>
<td>22.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>150 Freeze-Thaw Cycles</td>
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<td>G-FT-1</td>
<td>19.1</td>
<td>28.1</td>
<td>7.1</td>
</tr>
<tr>
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<td></td>
<td>2</td>
<td>G-FT-2</td>
<td>35.0</td>
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<td></td>
<td>3</td>
<td>G-FT-3</td>
<td>26.0</td>
<td></td>
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</tr>
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<td></td>
<td></td>
<td>4</td>
<td>G-FT-4</td>
<td>32.1</td>
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</tbody>
</table>

### Table 4.7: Maximum FRP strains achieved during pull-out testing

<table>
<thead>
<tr>
<th>Adhesive Type</th>
<th>Exposure Condition</th>
<th>Test Number</th>
<th>Specimen Name</th>
<th>Maximum FRP Strain (%)</th>
<th>Average (%)</th>
<th>Standard Deviation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy</td>
<td>Control</td>
<td>1</td>
<td>E-RT-1</td>
<td>1.15</td>
<td>1.20</td>
<td>0.05</td>
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<td></td>
<td></td>
<td>2</td>
<td>E-RT-2</td>
<td>1.17</td>
<td></td>
<td></td>
</tr>
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<td></td>
<td>3</td>
<td>E-RT-3</td>
<td>1.26</td>
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<td>4</td>
<td>E-RT-4</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>150 Freeze-Thaw Cycles</td>
<td>1</td>
<td>E-FT-1</td>
<td>0.96</td>
<td>0.91</td>
<td>0.10</td>
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<tr>
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<td></td>
<td>2</td>
<td>E-FT-2</td>
<td>0.81</td>
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<td></td>
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<tr>
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<td></td>
<td>3</td>
<td>E-FT-3</td>
<td>1.02</td>
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<td></td>
</tr>
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<td></td>
<td></td>
<td>4</td>
<td>E-FT-4</td>
<td>0.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grout</td>
<td>Control</td>
<td>1</td>
<td>G-RT-1</td>
<td>0.59</td>
<td>0.53</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>G-RT-2</td>
<td>0.65</td>
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<td></td>
<td></td>
<td>3</td>
<td>G-RT-3</td>
<td>0.42</td>
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<tr>
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<td>4</td>
<td>G-RT-4</td>
<td>0.47</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>150 Freeze-Thaw Cycles</td>
<td>1</td>
<td>G-FT-1</td>
<td>0.45</td>
<td>0.65</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>G-FT-2</td>
<td>0.83</td>
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<td>G-FT-3</td>
<td>0.61</td>
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<td>4</td>
<td>G-FT-4</td>
<td>0.73</td>
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</tr>
</tbody>
</table>
Figure 4.1: Peak loads for all slab strips and average loads for each specimen type

Figure 4.2: Peak FRP strains as a fraction of ultimate strain for each slab strip and average for each specimen type
Figure 4.3: Total applied load versus midspan deflection for all conditioning regimes for all grout adhesive strengthened slab strips

Figure 4.4: Midspan moment versus midspan curvature for all conditioning regimes for all grout adhesive strengthened slab strips
Figure 4.5: Midspan moment versus midspan FRP strain for all conditioning regimes for all grout adhesive strengthened slab strips.

Figure 4.6: Total applied load versus midspan deflection for all epoxy adhesive strengthened slab strips. Data taken from Burke (2008) are shown using dashed lines.
Figure 4.7: Midspan moment versus midspan curvature for all conditioning regimes for all epoxy adhesive strengthened slab strips. Data taken from Burke (2008) are shown using dashed lines.

Figure 4.8: Midspan moment versus midspan FRP strain for all conditioning regimes for the epoxy adhesive strengthened slab strips. Data taken from Burke (2008) are shown using dashed lines.
Figure 4.9: Total applied load versus midspan deflection for unconditioned slab strips tested at room temperature. Data taken from Burke (2008) are shown using dashed lines.

Figure 4.10: Moment versus curvature at midspan for unconditioned slab strips tested at room temperature. Data taken from Burke (2008) are shown using dashed lines.
Figure 4.11: Midspan moment versus FRP strain in the notch at midspan for unconditioned slab strips tested at room temperature. Data taken from Burke (2008) are shown using dashed lines.

Figure 4.12: Total applied load versus deflection at midspan for freeze-thaw cycled but unloaded slab strips.
Figure 4.13: Moment versus curvature at midspan for freeze-thaw cycled but unloaded slab strips

Figure 4.14: Moment versus FRP strain in the notch at midspan for freeze-thaw cycled but unloaded slab strips
Figure 4.15: Total applied load versus deflection at midspan for slab strips conditioned under sustained load at room temperature

Figure 4.16: Moment versus curvature at midspan for slab strips conditioned under sustained load at room temperature
Figure 4.17: Midspan moment versus FRP strain at midspan for slab strips conditioned under sustained load at room temperature

Figure 4.18: Total applied load versus midspan deflection for slab strips subjected to freeze-thaw cycles while under sustained load
Figure 4.19: Midspan moment versus midspan curvature for slab strips subjected to freeze-thaw cycles while under sustained load.

Figure 4.20: Midspan moment versus midspan FRP strain for slab strips subjected to freeze-thaw cycles while under sustained load.
Figure 4.21: Peak loads for all pull-out tests and average pull-out loads for each specimen type

Figure 4.22: Peak FRP strains as a fraction of ultimate strain for all pull-out tests and average for each specimen type
Figure 4.23: Load versus actuator stroke (less FRP elongation) for all room temperature pull-out tests

Figure 4.24: Load versus actuator stroke (less FRP elongation) for all grout adhesive pull-out tests
Figure 4.25: Load versus actuator stroke (less FRP elongation) for all epoxy adhesive pull-out tests
Figure 4.26: Typical grout strengthened slab strip failure at the FRP-grout interface (showing crumbled grout at a large flexural crack above the support location and essentially intact grout on either side of the crack)
Figure 4.27: Image of a typical failure of one of the epoxy adhesive strengthened slab strips just below the epoxy/concrete interface, again at the location of a major flexural crack over an internal support
Figure 4.28: Typical failure of a pull-out specimen for the grout adhesive system; the grout remains intact however the strip has slipped at the grout-strip interface
Figure 4.29: Typical failure of a pull-out specimen for the epoxy adhesive system; intact epoxy is visible around the FRP with a thin layer of concrete attached to the epoxy along the bond length. The bend in the FRP indicates that the strip has slipped upwards upon failure, forming the “s” shape due to the additional length between the specimen and the mechanical grips.
Chapter 5

Analysis

5.1 Introduction

This chapter presents analysis and discussion of the experimental results presented in Chapter 4. The chapter covers three related areas, including:

1. the development of a simple numerical model to predict the load-deformation response of the flexural specimens tested in the current project and comparison of the results against the experimental observations provided in Chapter 4 (Section 5.2);

2. the use of a previously developed bond model which is available in the literature, in conjunction with the aforementioned numerical model and in comparison against the experimental results given in Chapter 4 (Section 5.3); and

3. the attempted use of a unique high resolution digital image correlation algorithm to attempt track the FRP bond-slip response during the modified pull-out tests performed for the current study (Section 5.4).

5.2 Numerical Layer Model to Predict Flexural Response

To predict the flexural load-deformation response of the slab strips tested in the current project, a simple numerical layer model was created using Microsoft Excel spreadsheet software. The model essentially works by assuming perfect bond between all materials and a linear strain profile over the slab strips’ cross section at midspan and using a built-in goal seek function in MS Excel to vary the depth of the neutral axis, for a given compressive strain at the top of the section, until the total compressive forces balance the total tensile forces acting on the cross section. The
stress-strain relationships of the constituent materials are all included in the analysis. The model starts by assuming zero compressive strain at the top fibre of the concrete, and gradually increments the extreme compressive fibre strain (increasingly compressive) by the amount indicated in the “model inputs” worksheet. Strain increments of 0.00001 were found to be more than sufficient to completely map the moment-curvature response of the section, and since computational time was not an issue this value was used in all analyses. After each iteration the model would:

- increase the compressive strain at the extreme compressive fibre of the section;
- vary the location of the neutral axis (using the ‘goal seek’ function) until the corresponding compressive and tensile forces balanced over the cross section;
- calculate the corresponding bending moment, curvature, and FRP strain for the specific location of the neutral axis which balances the compressive and tensile forces (using a classical plane sections assumption); and
- export the bending moment, curvature, and FRP strain to the “model outputs” worksheet.

The model was set up such that the iterations of increasing compressive strain at the extreme fibre would continue until the compressive strain at the extreme fibre reached 0.0035. This was the assumed strain of concrete crushing based on typical assumptions used for concrete flexural design in Canada according to CSA A23.3-04 (CSA, 2006).

The dimensions of the slab strips’ cross-section as well as the material properties determined from ancillary testing were all entered into the “model inputs” worksheet of the model. In a “layer analysis” worksheet, the cross-section of the slab strip was divided into equally sized discrete layers. The thickness of each layer was set to 1 mm. This thickness was
chosen to be small enough to accurately predict the response of the slab strips, but large enough
to provide reasonably rapid computing run time. Furthermore, Burke (2008) performed a similar
analysis and varied the layer thickness of a numerical model of the same cross-section from 0.05
mm to 1 mm and noticed only minor effects on predicted behaviour (all within 1% for the
controls). Each type of tensile reinforcement (both the internal steel reinforcing bars and the
NSM FRP) was assumed to act at the location of the centroid of each respective material. Using
this assumption, the steel could be modelled at the layer through its centroid and the FRP could
be modelled at the layer which overlapped with its centroid.

The stress versus strain response of the steel was assumed to follow a modified Ramberg-
Osgood function as given by Ranger (2007) for precisely the same steel as was used in fabricating
the slab strips of the current study. This behaviour was incorporated into the model because the
cold drawn reinforcing wire did not display a clear yield point. Using a 0.2% offset strain
definition, the Young’s modulus and yield stress were determined to be 194 GPa and 667 MPa,
respectively. The equation for the stress-strain relationship is shown mathematically in Equation
5.2, and graphically in Figure 5.1.

\[
\begin{align*}
\sigma &= \varepsilon \left( \frac{1 - A}{1 + (B\varepsilon)^c} \right) \\
\sigma &= \varepsilon + \frac{(1 - 0.0078)}{(1 + (279.5\varepsilon)^{2.07})^{1/2.07}} \\
\end{align*}
\]

The assumed parabolic stress versus strain response curve for the concrete was taken
from Collins and Mitchell (1997). This behaviour is shown mathematically in Equation 5.3 and

graphically in Figure 5.2. The modulus of elasticity of the concrete (Equation 5.4) was determined by assuming a typical normal concrete density of 2400 kg/m$^3$. The strain in the concrete corresponding to peak stress was assumed to be 0.002. The concrete was assumed to remain uncracked while the tensile stress remained below the cracking stress, taken as $0.5\sqrt{f_c'}$.

\[ f_c' = \frac{n \left( \varepsilon_c' / \varepsilon_c \right)}{n - 1 + \left( \varepsilon_c' / \varepsilon_c \right)}^{n} \]

\[ E_c = \left( 3300 \sqrt{f_c'} + 6900 \right) \left( \varepsilon_c' / 2300 \right)^{1.5} \]

\[ n = 0.8 + \frac{f_c'}{17} \]

\[ k = 0.67 + \frac{f_c'}{62} \]

Where:

- $f_c'$ = compressive strength of concrete (MPa)
- $\varepsilon_c'$ = strain when $f_c$ reaches $f_c'$
- $\varepsilon_c$ = strain in the concrete
- $E_c$ = modulus of the concrete (MPa)
\( n \) = curve fitting factor

\( k \) = factor to increase post-peak decay in stress

\( \gamma_c \) = density of the concrete

The stress versus strain relationship of the FRP was taken from the tensile tests performed by Burke (2008) as already reported in Chapter 4. The FRP was assumed to be perfectly linear elastic until failure, with an ultimate stress of 2780 MPa, an ultimate strain of 2.0%, and a modulus of elasticity of 141 GPa. This is shown in Figure 5.3.

The numerical model was run until concrete crushing, with markers placed on the graph corresponding to the predicted failure strain using the Seracino et al. (2007a) model (discussed below) and the maximum tensile FRP strain allowed by ACI 440.2R-08 (ACI, 2008).

Results of the numerical model compared against the experimental data are shown in Figure 5.4 through Figure 5.11. The only difference between the freeze-thaw model and the room temperature model was the strength of the concrete. The room temperature model used a concrete strength of 41 MPa, while the freeze-thaw model used a concrete strength of 50 MPa. There are several differences between the model response and the experimental response. Most obviously, the model shows less stiffness prior to steel yielding (shown as the lower slope as compared to the experimental data). This is expected behaviour since the model does not take tension stiffening into account. The model, based on a representation of the cross-section, assumes the slab is uniformly cracked along the entire length once the tensile strain of the concrete is reached. In reality, however, the concrete would have carried some tensile load between the cracks. For a lightly reinforced section, such as these slabs, tension stiffening would likely have a greater impact on post cracking stiffness as compared to a heavily reinforced
section. An examination of the crack pattern after loading (see Figure 4.26 and Figure 4.27) confirms that the slabs were not uniformly cracked along the length, with the majority of the cracking occurring around the internal load points. This crack pattern is different than the behaviour expected by the model, and therefore could explain this difference in behaviour between the expected and experimental response.

There is also a discrepancy between the model and the test data at the time of concrete cracking. The model predicts a larger cracking moment than the experimental data with a subsequent drop in moment that was not observed in the laboratory. The model advances using small increments of compressive strain on the compressive face of the slab. Using this approach, after the initial concrete cracking, the moment is reduced while the model runs through a number of additional iterations gradually increasing the maximum compressive strain. In a real world situation, however, the slab response would be different because the load (and therefore moment) which caused the cracking would not disappear. The applied moment would remain constant while the curvature would increase until a point at which the equilibrium of forces was once again established. This could be forced in the model by running a horizontal line of increasing deformation from the point of cracking to the second ascending branch of the predicted response.

As expected, the model does predict the corresponding strain and curvature for the gradual yielding of the internal reinforcing steel reasonably well. The stiffness of the slab after steel yielding is also in close agreement with the test data as shown by the lines of similar slope. Overall, the model follows some of the trends of the experimental data and predicts some aspects of the behaviour of the slabs, confirming that a classical plane sections analysis approach is indeed appropriate for these NSM FRP strengthened reinforced concrete slab strips.
5.3 Bond Model to Predict Anchorage Strength

5.3.1 Slab Strip Debonding Model

As discussed in Chapter 2, bond models already exist in the literature that can be used to predict the intermediate crack induced (IC) debonding load in an externally-bonded or near surface mounted plate used to strengthen a concrete member. One such model, developed by Seracino et al. (2007a), is generic and can be used for predictions of both externally bonded and NSM FRP strengthening applications. Seracino’s model was derived from equations of equilibrium and plate-to-concrete compatibility. As previously discussed in Chapter 2, the equation below is assumed to give a lower bound bond strength for adhesively bonded plates.

\[ P_{IC} = \alpha_p 0.85\varphi_f 0.25 f'_{c}^{0.33} \sqrt{L_{per}(EA)_p} < f_{ult} A_p \quad \text{for FRP plates} \]
\[ f_y A_p \quad \text{for metallic plates} \]  

[Eq 5.7]

This analytical model can be used to predict the strain in an FRP plate at failure (in the case of the current thesis the plate is an NSM FRP strip). The values used in the model calculations to make predictions for the beams tested in the current thesis are summarized in Table 5.1 through Table 5.3.

Using the parameters outlined in Table 5.1 through Table 5.3, the room temperature debonding resistance for the case of an epoxy adhesive can be calculated according to the Seracino et al. (2007a) generic bond model as:

\[ P_{IC} = \alpha_p 0.85\varphi_f 0.25 f'_{c}^{0.33} \sqrt{L_{per}(EA)_p} < f_{ult} A_p \]  

[Eq 5.8]

\[ P_{IC} = (1.0)0.85(2.62)^{0.25}(41)^{0.33}\sqrt{(51.4)(141000)(31.2)} < (2780)(31.2) \]

\[ P_{IC} = 55.4 \text{ kN} < 86.7 \text{ kN} \]
Using this debonding load, the predicted room temperature debonding strain can be calculated as:

$$\varepsilon_{IC} = \varepsilon_{ult} \frac{P_{IC}}{(EA)_p} = 0.020 \frac{55.4}{86.7} = 0.0128$$

The predicted failure strain for the room temperature (unexposed) specimens with epoxy adhesive, when used in conjunction with the numerical layer model described in the previous section, is presented in Figure 5.4, where the predicted failure strain would result in an early cut-off of the predicted moment versus FRP strain response. For reference, the current ACI 440.2R-08 (ACI, 2008) strain limit for NSM FRP as flexural reinforcement is also presented in Figure 5.4. The Seracino et al. (2007a) model predicted a failure strain for the room temperature epoxy adhesive strengthened slab strips which was within 9% of the observed failure strain for all five tests. Although fairly close to the experimental data, the model was not consistently conservative. The ACI 440.2R-08 (ACI, 2008) strain limitation for flexural FRP in strengthening applications, however, was conservative for all cases. Indeed, if the section were designed using the ACI 440.2R-08 (ACI, 2008) strain limit the design would have been more than sufficient (by 11%) to resist the design loads. The midspan moment versus midspan curvature response for the room temperature epoxy adhesive strengthened slab strips is presented in Figure 5.5, showing a high degree of similarity between the model predictions and the test data.

Seracino et al. (2007a) noted that the failure of an FRP plate bonded to concrete is governed by the tensile strength of the concrete. The bond model assumes that the adhesive is sufficiently strong such that it forces the failure into the substrate concrete, and failure occurs when the principal stresses in the concrete exceed the tensile cracking strength of the concrete. Since the compressive strength (and therefore probably also the tensile strength) of the concrete
increased with freeze-thaw cycling (as noted from the ancillary test results on concrete described in Chapter 4), the theoretical debonding load of the NSM FRP strip also increased after freeze-thaw cycling. The calculations for the debonding load as well as the debonding strain for the slab strip after freeze-thaw cycling are thus the same as above, except for this change in concrete strength. The post-freeze-thaw cycling predicted intermediate crack induced debonding load and strain are 59.1 kN and 1.36%, respectively, for the epoxy adhesive strengthened slab strips.

The midspan moment versus midspan FRP strain observations for the freeze-thaw cycled epoxy adhesive slab strips are contrasted against the numerical layer model in Figure 5.6. The layer model in this case assumes the stronger compressive concrete strength of 50 MPa as compared to the room temperature compressive strength of 41 MPa. Included on the graph are the predicted failure strains using the Seracino et al. (2007a) model as well as the maximum tensile strain permitted by ACI 440.2 (ACI, 2008). The Seracino et al. (2007a) model over predicts the failure strain for these specimen from 14% to 26%. Although less severe, the ACI strain limit also exceeded all epoxy freeze-thaw tests. A similar graph was also developed to compare the midspan moment versus midspan curvature response for the post freeze-thaw cycled epoxy slab strips (Figure 5.7).

To apply the Seracino et al. (2007a) model to the grout slab strips, the failure plane was assumed to move inward, from the concrete into the adhesive. Using this assumption, both the length and width of the failure plane were reduced. Rather than the failure U-shaped failure plane in the concrete adjacent to the epoxy, the failure was assumed to occur right at the FRP/grout interface (as was observed in testing). The length and width of the failure plane were assumed to be the length and width of the FRP strip. The aspect ratio deals with the shape of the failure plane, and accounts for the amount of confinement provided by the specific failure plane. It is
large for NSM systems, and smaller for EB systems. Although the length of the failure plane decreases for the case of the grout adhesive, the increase in the aspect ratio more than reverses the effect of the smaller failure plane, actually predicting failure strains which are greater than the epoxy slab strips. Using the parameters outlined in Table 5.1 through Table 5.4, the room temperature debonding resistance for the case of a grout adhesive can be calculated according to the Seracino et al. (2007a) generic bond model as:

\[
P_{IC} = \alpha_p 0.85 \phi_f^{0.25} f_c^{0.33} \sqrt{L_{per}(EA)_p} < f_{ult} A_p
\]

Using the debonding load, the predicted room temperature debonding strain can be calculated as:

\[
\varepsilon_{IC} = \frac{P_{IC}}{(EA)_p} = 0.020 \frac{59.5}{86.7} = 0.0137
\]

Post freeze-thaw cycling, the only factor which changes is the concrete strength. The post freeze-thaw cycling grout adhesive debonding load predicted by the Seracino et al. (2007a) model is 63.6 kN with a corresponding debonding strain of 1.47%.

The actual and predicted failure strains for the slab-strip testing program are summarized in Table 5.5. It should be noted that although the model was used to predict the failure strains of the members with grout adhesive, the model was clearly not designed for this purpose. The model assumes the adhesive is sufficiently strong such that the failure occurs in the concrete. In all cases, however, the FRP in the grout adhesive failed at the FRP-grout interface. There are parameters in the model which account for the location of the failure plane, however, Seracino et
al. (2007a) noted that these parameters have little if any affect on the performance of the model. For these reasons, the model predicted failure strains for the grout slab strips are vastly different from the results obtained during testing. This model should not be used for grout adhesive NSM applications.

The model predicted failure strains for the room temperature and freeze-thaw cycled grout adhesive slab strips are shown in Figure 5.8 and Figure 5.10. The room temperature grout debonding strains are over predicted by the model between 23% and 41%. After freeze-thaw cycling, the model over predicts the FRP strain at which debonding occurs anywhere from 53% to 83%. The midspan moment versus midspan curvatures for the grout adhesive slab strips are shown in Figure 5.9 and Figure 5.11 with excellent match between the model and the non-sustained load results, with the exception of the model assuming full composite action in the FRP until concrete crushing. The sustained load slab strips, as discussed in Chapter 4, produced very different midspan moment versus midspan curvature results, potentially from large flexural cracks which were initiated during the period of sustained load.

5.3.2 Pull-Out Debonding Model

To predict the failure strains of the pull-out tests, another Seracino et al (2007b) model was used (Equation 5.10). The benefit of this model being that it takes bond length into account when predicting the intermediate crack induced debonding load and that it was developed from nonlinear statistical analysis on 36 push-pull tests similar to the pull-out tests of the current thesis. It should be noted that for all 36 of Seracino et al.’s (2007b) tests, failure occurred in the concrete rather than in the adhesive.
This model was not used to predict the failure loads of the slab strips because it was developed using a mathematical fit for a specific series of push-pull tests which are more similar to the bond tests performed in this thesis than the slab strips. The generic debonding model, in contrast, was derived considering equilibrium and compatibility of the plate-to-concrete joint. Using the statistical model in Equation 5.10, the room temperature debonding load can be calculated as:

\[ P_{IC} = \alpha \beta \sqrt{f_c d_p^{1.36} b_p^{0.21}} \leq f_{rupt} b_p d_p \]  

where:

\[ \alpha = \begin{cases} 0.19 \text{ for mean value} \\ 0.16 \text{ for characteristic value} \end{cases} \]

\[ \beta = \begin{cases} 1.0 \text{ for } L \geq 200 \text{ mm} \\ \frac{L}{200} \text{ for } L < 200 \text{ mm} \end{cases} \]

\[ d_p = \text{dimension of the FRP plate parallel to the concrete surface (mm)} \]

\[ b_p = \text{dimension of the FRP plate perpendicular to the concrete surface (mm)} \]

\[ f_{rupt} = \text{rupture stress of FRP (MPa)} \]

This model was not used to predict the failure loads of the slab strips because it was developed using a mathematical fit for a specific series of push-pull tests which are more similar to the bond tests performed in this thesis than the slab strips. The generic debonding model, in contrast, was derived considering equilibrium and compatibility of the plate-to-concrete joint. Using the statistical model in Equation 5.10, the room temperature debonding load can be calculated as:

\[ P_{IC} = \alpha \beta \sqrt{f_c d_p^{1.36} b_p^{0.21}} \leq f_{rupt} b_p d_p \]

\[ P_{IC} = (0.19) \left( \frac{150}{200} \right) \sqrt{41(16)^{1.36}(2)^{0.21}} \leq (2780)(2)(16) \]

\[ P_{IC} = 45.8 \text{ kN} \leq 88.9 \text{ kN} \]

The debonding strain corresponding to 45.8 kN can be calculated as:
Again, for the case of the debonding after freeze-thaw cycling, the only thing that changes is the strength of the concrete. Using the same equation, the intermediate crack induced debonding load of the pull-out specimens after freeze-thaw cycling is 51.1 kN. Similarly, the intermediate crack induced debonding strain is 1.15%. The bond model is conservative for the case of the room temperature epoxy strengthened pull-outs. After freeze-thaw cycling, however, the model over predicts failure strain by 26%. Although the model was not created for grout adhesives, it was also compared against the grout pull-out tests performed in this thesis. The results of the predicted and observed FRP failure strains normalized against ultimate FRP strain are displayed in Figure 5.12. The model produces failure strains for the grout pull-out specimens which are more than double the observed failure strains. The discrepancy between the test results and the model for the grout adhesive highlights the different failure mechanism, and suggests that the Seracino et al. (2007b) model should only be applied with epoxy adhesives. This is not unexpected, as all 36 push-pull tests used in the generation of this model experienced failure in the concrete.

5.4 Digital Image Correlation Analysis: Bond-Slip Response

Attempts were made to use an advanced digital image correlation algorithm to monitor the bondslip response of the NSM FRP strips during the modified pullout testing, as described in Chapter 4. This was not particularly successful, mostly due to the failure mode of the pullout specimens which prevented accurate image correlation. However, the details of the technique and the results of a few successful analyses are presented in this section.
Digital image correlation analysis is a process by which high resolution digital photos are taken at regular intervals during a test, and these are then used to track selected in-plane movements within the field of view of the photo. The method tracks specific pixel ‘patches’ within the field of view through a series of photos taken at constant intervals. By tracking relative movements on a material, this approach can be used to calculate strains. By tracking movements relative to a stationary reference, this approach can be used to calculate displacements. For the reader who would like to delve into the specific details of the digital image analysis, White et al. (2003) explain the theory and validate the performance of this novel measurement technique.

For the purposes of this thesis, a high resolution digital camera was set up in front of each pull-out bond test prior to testing. A halogen light was placed behind the camera to illuminate the pull-out specimen to provide clear contrast within the images. Several images were taken before the start of the test to ensure the camera was aligned properly and was accurately zoomed on the area of interest (the loaded end of the bonded length and the FRP immediately above the bonded length). A sample image from one of the pull-out tests is shown in Figure 5.13. The image needed to provide a large enough field of view to encapsulate two pixel patches: one on the FRP strip located 25 mm above the edge of the concrete, and a second located on the recessed face of the pull-out specimen away from the bond line. The second pixel patch was chosen to provide a stationary reference to calculate the movement (slip) of the FRP strip at the loaded end relative to the pull-out concrete. A scale was also included in the photos to determine the absolute size of a pixel. Initially, the movement of the FRP was calculated in pixels and then later converted into displacements in mm using the scale within the image. The camera, which was connected to a laptop computer, took a photo every five seconds during testing so that the images could later be
correlated to the applied load. Knowing the time interval between each photo and the fact that the first photo was taken at time zero, the image analysis could then be used to compare against the traditional data collected during testing (i.e. load, stroke, LP, etc).

Initially, the digital photos were intended to be used to measure the loaded end slip for all pull-out tests. To track movements in the photos, however, a stationary reference point is needed. As mentioned during the results chapter, testing of the epoxy pull-out tests involved considerable cracking in the concrete surrounding the loaded end of the bonded length. Indeed, in most cases where epoxy adhesive was used diagonal cracking of the concrete occurred early in the tests and a wedge of concrete completely popped-off during debonding failure. Because essentially all of the concrete in the frame of view of the photo analysis images was gradually moving upwards during these tests, with the concrete wedge moving in a rigid-body motion attached to the strip, it quickly became obvious that it would not be possible to develop a loaded-end load versus bond-slip response for the epoxy adhesive bond pull-out specimens. The grout adhesive pull-out specimens, on the other hand, failed by slipping of the FRP at the FRP/grout interface, without much cracking or movement of the concrete surrounding the bond. For this reason, it was possible to use the high resolution digital images to create a load versus slip graph (Figure 5.14). The bond slip graph displays a linear branch followed by a softening branch. This behaviour confirms that the apparent ‘stiffening’ observed on the load versus actuator stroke graphs in Chapter 4 was indeed the seating of the wedge action grips. The load versus slip figure displays no discernable differences between the freeze-thaw conditioned pull-out specimens and the controls, confirming that freeze-thaw cycling does not obviously adversely impact the bond performance of grout adhesives in NSM situations.
Table 5.1: Material properties used in the model

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength after room temperature exposure</td>
<td>$f'_{c, rt}$</td>
<td>41 MPa</td>
</tr>
<tr>
<td>Concrete compressive strength after freeze-thaw cycling</td>
<td>$f'_{c, ft}$</td>
<td>50 MPa</td>
</tr>
<tr>
<td>Modulus of elasticity of the FRP strip</td>
<td>$E_p$</td>
<td>141000 MPa</td>
</tr>
<tr>
<td>Rupture strength of the FRP strip</td>
<td>$f_{ult}$</td>
<td>2780 MPa</td>
</tr>
<tr>
<td>Cross-sectional area of the FRP strip</td>
<td>$A_p$</td>
<td>31.2 mm$^2$</td>
</tr>
</tbody>
</table>

Table 5.2: Assumed properties used in the model

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to failure plane in concrete (distance from adhesive to failure plane)</td>
<td>$t_b, t_d$</td>
<td>1 mm</td>
</tr>
<tr>
<td>Bond parameter (set to 1.0 to determine the mean value)</td>
<td>$\alpha_p$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 5.3: NSM groove properties used in the epoxy slab strip model

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of failure plane = depth of groove + $t_d$</td>
<td>$d_f$</td>
<td>22 mm</td>
</tr>
<tr>
<td>Width of failure plane = width of groove +2 $t_b$</td>
<td>$b_f$</td>
<td>8.4 mm</td>
</tr>
<tr>
<td>Length of failure perimeter = 2$d_f + b_f$</td>
<td>$L_{per}$</td>
<td>51.4 mm</td>
</tr>
<tr>
<td>Aspect ratio $\frac{d_f}{b_f}$</td>
<td>$\Phi_f$</td>
<td>2.62</td>
</tr>
</tbody>
</table>

Table 5.4: NSM groove properties used in the grout slab strip model

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of failure plane = depth of strip</td>
<td>$d_f$</td>
<td>16 mm</td>
</tr>
<tr>
<td>Width of failure plane = width of strip</td>
<td>$b_f$</td>
<td>2 mm</td>
</tr>
<tr>
<td>Length of failure perimeter = 2$d_f + b_f$</td>
<td>$L_{per}$</td>
<td>34 mm</td>
</tr>
<tr>
<td>Aspect ratio $\frac{d_f}{b_f}$</td>
<td>$\Phi_f$</td>
<td>8</td>
</tr>
</tbody>
</table>
Table 5.5: Observed and predicted FRP failure loads and strains for the slab strip testing program

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimen Name</th>
<th>Failure Load (kN)</th>
<th>FRP Strain at Failure (%)</th>
<th>Predicted FRP Strain at Failure (%)</th>
<th>Predicted failure strain Actual failure strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>E-RT-1</td>
<td>no data*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>E-RT-2</td>
<td>32.6</td>
<td>1.31</td>
<td>1.28</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>E-RT-B1</td>
<td>33.5</td>
<td>1.26</td>
<td>1.28</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>E-RT-B2</td>
<td>34.9</td>
<td>1.41</td>
<td>1.28</td>
<td>0.91</td>
</tr>
<tr>
<td>3</td>
<td>G-RT-1</td>
<td>27.2</td>
<td>0.97</td>
<td>1.37</td>
<td>1.41</td>
</tr>
<tr>
<td>4</td>
<td>G-RT-2</td>
<td>28.4</td>
<td>0.98</td>
<td>1.37</td>
<td>1.40</td>
</tr>
<tr>
<td>5</td>
<td>G-RT-3</td>
<td>28.0</td>
<td>0.99</td>
<td>1.37</td>
<td>1.38</td>
</tr>
<tr>
<td>6</td>
<td>E-FT-1</td>
<td>33.1</td>
<td>1.14</td>
<td>1.36</td>
<td>1.19</td>
</tr>
<tr>
<td>7</td>
<td>E-FT-2</td>
<td>32.6</td>
<td>1.19</td>
<td>1.36</td>
<td>1.14</td>
</tr>
<tr>
<td>8</td>
<td>G-FT-1</td>
<td>29.5</td>
<td>0.90</td>
<td>1.47</td>
<td>1.63</td>
</tr>
<tr>
<td>9</td>
<td>G-FT-2</td>
<td>27.7</td>
<td>0.91</td>
<td>1.47</td>
<td>1.62</td>
</tr>
<tr>
<td>10</td>
<td>G-FT-3</td>
<td>27.6</td>
<td>0.93</td>
<td>1.47</td>
<td>1.58</td>
</tr>
<tr>
<td>11</td>
<td>E-SL-1</td>
<td>32.8</td>
<td>1.26</td>
<td>1.28</td>
<td>1.02</td>
</tr>
<tr>
<td>12</td>
<td>E-SL-2</td>
<td>33.3</td>
<td>1.21</td>
<td>1.28</td>
<td>1.06</td>
</tr>
<tr>
<td>13</td>
<td>E-SL-3</td>
<td>no data**</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>G-SL-1</td>
<td>29.9</td>
<td>0.99</td>
<td>1.37</td>
<td>1.38</td>
</tr>
<tr>
<td>15</td>
<td>G-SL-2</td>
<td>30.1</td>
<td>1.08</td>
<td>1.37</td>
<td>1.27</td>
</tr>
<tr>
<td>16</td>
<td>G-SL-3</td>
<td>31.6</td>
<td>1.11</td>
<td>1.37</td>
<td>1.23</td>
</tr>
<tr>
<td>17</td>
<td>E-FT-SL-1</td>
<td>32.0</td>
<td>1.08</td>
<td>1.36</td>
<td>1.26</td>
</tr>
<tr>
<td>18</td>
<td>E-FT-SL-2</td>
<td>30.0</td>
<td>1.08</td>
<td>1.36</td>
<td>1.26</td>
</tr>
<tr>
<td>19</td>
<td>G-FT-SL-1</td>
<td>29.2</td>
<td>0.96</td>
<td>1.47</td>
<td>1.53</td>
</tr>
<tr>
<td>20</td>
<td>G-FT-SL-2</td>
<td>27.6</td>
<td>0.80</td>
<td>1.47</td>
<td>1.83</td>
</tr>
<tr>
<td>21</td>
<td>G-FT-SL-3</td>
<td>28.8</td>
<td>0.92</td>
<td>1.47</td>
<td>1.60</td>
</tr>
</tbody>
</table>

* A data acquisition system malfunction rendered the acquired data useless
** The specimen failed during handling while it was being loaded into the test setup
The two tests listed in Italics were performed by Burke (2008) and are included for comparison purposes
Table 5.6: Observed and predicted FRP failure strains for the pull-out testing program

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimen Name</th>
<th>Failure Load (kN)</th>
<th>FRP Strain at Failure (%)</th>
<th>Predicted FRP Strain at Failure (%)</th>
<th>Predicted failure strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>E-RT-1</td>
<td>50.6</td>
<td>1.15</td>
<td>1.03</td>
<td>0.90</td>
</tr>
<tr>
<td>2</td>
<td>E-RT-2</td>
<td>52.2</td>
<td>1.17</td>
<td>1.03</td>
<td>0.88</td>
</tr>
<tr>
<td>3</td>
<td>E-RT-3</td>
<td>55.4</td>
<td>1.26</td>
<td>1.03</td>
<td>0.82</td>
</tr>
<tr>
<td>4</td>
<td>E-RT-4</td>
<td>55.7</td>
<td>1.22</td>
<td>1.03</td>
<td>0.84</td>
</tr>
<tr>
<td>5</td>
<td>G-RT-1</td>
<td>40.9</td>
<td>0.96</td>
<td>1.03</td>
<td>1.07</td>
</tr>
<tr>
<td>6</td>
<td>G-RT-2</td>
<td>34.5</td>
<td>0.81</td>
<td>1.03</td>
<td>1.27</td>
</tr>
<tr>
<td>7</td>
<td>G-RT-3</td>
<td>43.6</td>
<td>1.02</td>
<td>1.03</td>
<td>1.01</td>
</tr>
<tr>
<td>8</td>
<td>G-RT-4</td>
<td>36.5</td>
<td>0.84</td>
<td>1.03</td>
<td>1.23</td>
</tr>
<tr>
<td>9</td>
<td>E-FT-1</td>
<td>32.2</td>
<td>0.59</td>
<td>1.15</td>
<td>1.95</td>
</tr>
<tr>
<td>10</td>
<td>E-FT-2</td>
<td>28.2</td>
<td>0.65</td>
<td>1.15</td>
<td>1.77</td>
</tr>
<tr>
<td>11</td>
<td>E-FT-3</td>
<td>16.4</td>
<td>0.42</td>
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<tr>
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<td>E-FT-4</td>
<td>22.9</td>
<td>0.47</td>
<td>1.15</td>
<td>2.45</td>
</tr>
<tr>
<td>13</td>
<td>G-FT-1</td>
<td>19.1</td>
<td>0.45</td>
<td>1.15</td>
<td>2.56</td>
</tr>
<tr>
<td>14</td>
<td>G-FT-2</td>
<td>35.0</td>
<td>0.83</td>
<td>1.15</td>
<td>1.39</td>
</tr>
<tr>
<td>15</td>
<td>G-FT-3</td>
<td>26.0</td>
<td>0.61</td>
<td>1.15</td>
<td>1.89</td>
</tr>
<tr>
<td>16</td>
<td>G-FT-4</td>
<td>32.1</td>
<td>0.73</td>
<td>1.15</td>
<td>1.58</td>
</tr>
</tbody>
</table>
Figure 5.1: Stress versus strain profile used for the reinforcing steel in the numerical layer model

Figure 5.2: Stress versus strain profiles for the concrete used in the numerical layer model
Figure 5.3: Stress versus strain profile used for the FRP in the numerical layer model

Figure 5.4: Midspan moment versus FRP strain at midspan from room temperature epoxy adhesive strengthened slab strips contrasted with the predictions of the numerical layer model
Figure 5.5: Midspan moment versus midspan curvature data from room temperature epoxy adhesive strengthened slab strips contrasted with the predictions of the numerical layer model.

Figure 5.6: Midspan moment versus midspan FRP strain for the epoxy adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model.
Figure 5.7: Midspan moment versus midspan curvature for the epoxy adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model.

Figure 5.8: Midspan moment versus midspan FRP strain for the grout adhesive strengthened slab strips contrasted with the predictions of the numerical layer model.
Figure 5.9: Midspan moment versus midspan curvature for the grout adhesive strengthened slab strips contrasted with the predictions of the numerical layer model.

Figure 5.10: Midspan moment versus midspan FRP strain for the grout adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model.
Figure 5.11: Midspan moment versus midspan curvature for the grout adhesive strengthened slab strips subjected to 300 freeze-thaw cycles contrasted with the predictions of the numerical layer model.

Figure 5.12: Peak FRP strains as a fraction of ultimate strain for all pull-out tests and average for each specimen type compared against the Seracino et al. (2007b) model predicted debonding strains.
Figure 5.13: Sample high resolution digital photo taken for the digital image correlation analysis. One pixel patch was placed on the FRP 25 mm above the top of the concrete block, and another patch was placed in the bottom left corner as a zero displacement reference.

Figure 5.14: Applied load versus loaded end slip data from high resolution digital image analysis for the grout pull-out specimens
Chapter 6

Conclusions and Recommendations

6.1 Overview

The research program presented in this thesis in many respects represents a logical extension of prior research programs which have been conducted at Queen’s University studying the freeze-thaw durability of FRP strengthened concrete members. This specific study focused on the durability of a near surface mounted carbon FRP strip strengthening system in a series of flexural tests (21 slab strips) and bond tests (16 pull-out tests). Both epoxy and cementitious adhesive systems have been investigated.

The slab strip testing program investigated NSM FRP strengthened reinforced concrete member performance after exposure to one of four exposure regimes: (1) exposure to room temperature (control specimens), (2) exposure to 300 freeze-thaw cycles, (3) exposure to a period of sustained load, and (4) exposure to 300 freeze-thaw cycles while under sustained load. The pull-out testing program investigated bond performance after: (1) exposure to room temperature (control specimens), or (2) exposure to 150 freeze-thaw cycles. Conclusions from the testing program are highlighted in the following section.

6.2 Conclusions

Based on the experimental results presented in the current thesis, the following primary conclusions can be drawn:

1. Although the epoxy adhesive provided super bond performance (as displayed by greater ultimate loads and strains in the FRP), the grout adhesive can still be used provided more stringent strain limits are imposed (in the neighbourhood of 40% to 50% of ultimate FRP
strain based on the current testing program). New bond models are needed which can accurately predict the debonding load for grout adhesive systems. Despite this, grout adhesives seem to be particularly attractive for external strengthening applications because freeze-thaw cycles and/or sustained load do not appear to negatively impact the ultimate capacity of the bond.

2. A period of sustained load and/or 300 freeze-thaw cycles did not appear to drastically alter the behaviour of an NSM FRP flexural strengthening systems using epoxy adhesive, although minor reductions in ultimate load (in the range of 2% to 8%) were observed.

3. It would appear that pull-out bond tests suffer proportionally more freeze-thaw damage when using an epoxy adhesive. After 150 freeze-thaw cycles, the bond tests with epoxy adhesive displayed more reduction in ultimate load (27%) as compared to the flexurally strengthened slab strips exposed to freeze-thaw cycles (2%). This issue clearly requires additional study.

6.3 Recommendations

The research program presented in this thesis has sought to investigate the freeze-thaw and/or sustained load durability of NSM FRP strengthening systems for concrete structures. A number of areas where additional research is warranted have been identified. Specifically, more work is needed to:

- examine the freeze-thaw performance of epoxy adhesives in NSM FRP bond tests and shear strengthening scenarios to determine if the load losses observed in this testing program represent typical values and to determine if the observed bond strength degradation is proportional to the number of freeze-thaw cycles and if it is a legitimate concern for NSM FRP strengthening systems applied to real structures;
• examine the potential bond degradation for epoxy adhesives used for NSM FRP in conditions of freeze-thaw cycles while under sustained load;

• determine defensible and conservative strain limits for NSM FRP strengthening systems installed using cementitious grout adhesives in both flexural and shear strengthening applications; and

• produce models which can more accurately predict FRP debonding strains for grout adhesives so that they can be safely used as a more economical alternative to epoxy adhesives in NSM FRP strengthening applications.
References

Anonymous Concrete Technology > Durability > Freeze-thaw Resistance

Anonymous Concrete Basics Home > Air-Entrained Concrete


Hobbs D. Concrete deterioration: Causes, diagnosis, and minimising risk. 2001; .


Hu A. Experimental Study of Effects of Freeze-Thaw Cycle on Concrete Columns Strengthened with FRP Laminates. da lian li gong da xue xue bao. 2006; 46(6):862.


Appendix A

Slab Strip Design Calculations
The flexural and shear strength of the slab strips were calculated with both the American and Canadian codes. The following appendix highlights the calculations which were performed.

**Flexural Strength**

**Material Properties**

<table>
<thead>
<tr>
<th></th>
<th>Concrete</th>
<th>Steel</th>
<th>FRP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CSA</td>
<td>ACI</td>
<td></td>
</tr>
<tr>
<td>$E_c$</td>
<td>$E_c = 4500 \sqrt{f'_c}$</td>
<td>$E_c = 4732 \sqrt{f'_c}$</td>
<td>$E_s = 194$ GPa</td>
</tr>
<tr>
<td>$= 30190$ MPa</td>
<td>$= 31740$ MPa</td>
<td>$E_f = 124$ GPa</td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{ult}$</td>
<td>$0.0035$</td>
<td>$0.003$</td>
<td>$\varepsilon_{fult} = 0.017$</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>$45$ MPa</td>
<td>$45$ MPa</td>
<td>$f_y = 693$ MPa</td>
</tr>
<tr>
<td>$A_s$</td>
<td>$64.3$ mm$^2$</td>
<td>$A_{frp} = 31.2$ mm$^2$</td>
<td></td>
</tr>
<tr>
<td>$d_s$</td>
<td>$71$ mm</td>
<td>$d_{frp} = 91$ mm</td>
<td></td>
</tr>
</tbody>
</table>

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165
ACI 440.2R-08 (ACI, 2008) and ACI 318-05 (ACI, 2005)

Flexural Strength of Unstrengthened Slab-Strip

\[ C = T \]

\[ C = 0.85 f'_c \beta_1 c b \]

\[ \beta_1 = \begin{cases} 0.85 & \text{if } f'_c < 4000 \text{ psi (27.6 MPa)} \\ 0.85 - 0.05 \left( \frac{f'_c - 4000}{1000} \right) & \text{if } f'_c > 4000 \text{ psi (27.6 MPa)} \\ 0.85 - 0.05 \left( \frac{f'_c - 27.58}{6.89} \right) & \text{if } f'_c > 27.6 \text{ MPa (same as above)} \end{cases} \]

\[ T_s = A_s f_s \]

Assume failure is caused by concrete crushing after yielding of the internal steel reinforcement.

\[ \beta_1 = 0.85 - 0.05 \left( \frac{45 \text{MPa} - 27.58 \text{MPa}}{6.89 \text{MPa}} \right) = 0.72 \]

\[ C = 0.85(45 \text{MPa})(0.72)c(254 \text{mm}) = 6995c \text{ N} \]

\[ T_s = (64.3 \text{mm}^2)(693 \text{MPa}) = 44560 \text{ N} \]

By setting the compression force equal and opposite to the tension force, \( c = 6.37 \text{ mm} \)

\[ \varepsilon_{u_{lt}} = \varepsilon_{u_{lt}} \left( \frac{d_s - c}{c} \right) = 0.003 \left( \frac{71 - 6.37}{6.37} \right) = 0.0304 > \varepsilon_{s_y} = 0.0036 \quad \therefore \text{steel has yielded} \]

\[ a = \beta_1 c = (0.72)(6.37 \text{mm}) = 4.59 \text{mm} \]
\[ M_r = A_s f_y \left( d_s - \frac{a}{2} \right) = \left( 64.3 \, \text{mm}^2 \right) \left( 693 \, \text{MPa} \right) \left( 71 \, \text{mm} - \frac{4.59 \, \text{mm}}{2} \right) \]

\[ M_r = 3061568 \, \text{Nm} = 3.06 \, \text{kNm} \]
Flexural Strength of Strengthened Slab-Strip

\[ C = T \]

\[ C = 0.85 f' c \beta_1 c \]

\[ \beta_1 = \begin{cases} 
0.85 & \text{if } f' c < 4000 \text{ psi (27.6 MPa)} \\
0.85 - 0.05 \left( \frac{f' c - 4000}{1000} \right) & > 0.65 \text{ if } f' c > 4000 \text{ psi (27.6 MPa)} \\
0.85 - 0.05 \left( \frac{f' c - 27.58}{6.89} \right) & > 0.65 \text{ if } f' c > 27.6 \text{ MPa (same as above)}
\end{cases} \]

\[ T_s = A_{sf} \]

\[ T_{frp} = A_{frp} f_{frp} \]

\[ \varepsilon_{frp} = \varepsilon_{ult} \left( \frac{d_{frp} - c}{c} \right) \]

Assume failure is caused by concrete crushing after yielding of the internal steel reinforcement but prior to the debonding or rupture of the FRP strip.

\[ \beta_1 = \left( 0.85 - 0.05 \left( \frac{45 \text{ MPa} - 27.58 \text{ MPa}}{6.89 \text{ MPa}} \right) = 0.72 \right. \]

\[ C = 0.85(45 \text{ MPa})(0.72)c(254 \text{ mm}) = 6995c \text{ N} \]

\[ T_s = (64.3 \text{ mm}^2)(693 \text{ MPa}) = 44560 \text{ N} \]

\[ T_{frp} = A_{frp} E_{frp} \varepsilon_{frp} = A_{frp} E_{frp} \varepsilon_{ult} \left( \frac{d_{frp} - c}{c} \right) \]
\[ T_{frp} = (31.2 \text{mm}^2)(124000 \text{MPa})(0.003) \left( \frac{91 \text{mm} - c}{c} \right) \]

By setting the compression force equal and opposite to the tension force, \( c = 14.87 \text{ mm} \)

\[ \varepsilon_{ult} = \varepsilon_{ult} \left( \frac{d_s - c}{c} \right) = 0.003 \left( \frac{71 - 14.87}{14.87} \right) = 0.0113 > \varepsilon_{sy} = 0.0036 \quad \therefore \text{steel has yielded} \]

\[ \varepsilon_{frp} = \varepsilon_{ult} \left( \frac{d_{frp} - c}{c} \right) = 0.003 \left( \frac{91 \text{mm} - 14.87 \text{mm}}{14.87 \text{mm}} \right) = 0.0154 < \varepsilon_{fult} = 0.017 \]

\( \therefore \text{frp has not ruptured} \)

\[ a = \beta_1 c = (0.72)(14.87 \text{mm}) = 10.71 \text{mm} \]

\[ M_r = A_s f_y \left( d_s - \frac{a}{2} \right) + A_{frp} E_{frp} \varepsilon_{frp} \left( d_{frp} - \frac{a}{2} \right) \]

\[ M_r = A_s f_y \left( d_s - \frac{a}{2} \right) + A_{frp} E_{frp} \varepsilon_{frp} \left( d_{frp} - \frac{a}{2} \right) \]

\[ M_r = (64.3 \text{mm}^2)(693 \text{MPa}) \left( 71 \text{mm} - \frac{10.71 \text{mm}}{2} \right) \]

\[ + (31.2 \text{mm}^2)(124000 \text{MPa})(0.0154) \left( 91 \text{mm} - \frac{10.71 \text{mm}}{2} \right) \]

\[ M_r = 8.03 \text{ kNm} \]

ACI 440.2R-07 limits the strain in the FRP strip to 70% of ultimate to prevent bond failure

\[ \varepsilon_{frp} < 0.7 \varepsilon_{fult} \]

\[ \varepsilon_{frp} < 0.7(0.017) = 0.0119 \]
When the strain in the FRP is 0.0119, the concrete has not yet crushed.

\[ C = 0.85E_c\varepsilon_c\beta_1 cb \]

\[ C = 0.85E_c(e_{frp})\left(\frac{c}{d_{frp}-c}\right)\beta_1 cb \]

\[ T_s = A_s f_s \]

\[ T_{frp} = A_{frp} E_{frp}\varepsilon_{frp} \]

\[ C = 0.85(31740\text{MPa})(0.0119)\left(\frac{91\text{mm} - c}{91\text{mm} - c}\right)(0.72)c(254\text{mm}) \]

\[ T_s = (64.3\text{mm}^2)(693\text{MPa}) = 44560\ N \]

\[ T_{frp} = (31.2\text{mm}^2)(124000\text{MPa})(0.0119) = 46039\ N \]

By setting the compression force equal and opposite to the tension force, \( c = 11.10 \text{ mm} \)

\[ \varepsilon_s = \varepsilon_{frp}\left(\frac{d_s - c}{d_{frp} - c}\right) = 0.0119\left(\frac{71 - 11.1}{91 - 11.1}\right) = 0.0089 \Rightarrow \varepsilon_{sy} = 0.0036 \Rightarrow \text{steel has yielded} \]

\[ \varepsilon_c = \varepsilon_{frp}\left(\frac{c}{d_{frp} - c}\right) = 0.0119\left(\frac{11.1\text{mm}}{91\text{mm} - 11.1\text{mm}}\right) = 0.00147 < \varepsilon_{ult} = 0.003 \]

\( \Rightarrow \) concrete has not crushed

\[ a = \beta_1 c = (0.72)(11.1\text{mm}) = 7.99\text{mm} \]

\[ M_r = A_s f_y \left(d_s - \frac{a}{2}\right) + A_{frp} E_{frp}\varepsilon_{frp} \left(d_{frp} - \frac{a}{2}\right) \]
\[ M_r = (64.3\text{mm}^2)(693\text{MPa})\left(71\text{mm} - \frac{7.99\text{mm}}{2}\right) \\
+ (31.2\text{mm}^2)(124000\text{MPa})(0.0119)\left(91\text{mm} - \frac{7.99\text{mm}}{2}\right) \]

\[ M_r = 6.99 \text{ kNm} \]
ACI 440.2R-08 (ACI, 2008) and CSA A23.3-04 (CSA, 2004)

Flexural Strength of Unstrengthened Slab-Strip

\[ C = T \]

\[ C = \alpha_1 f'_c \beta_1 c b \]

\[ \alpha_1 = 0.85 - 0.0015 f'_c \geq 0.67 \]

\[ \beta_1 = 0.97 - 0.0025 f'_c \geq 0.67 \]

\[ T_s = A_{fs} \]

Assume failure is caused by concrete crushing after yielding of the internal steel reinforcement.

\[ \alpha_1 = 0.85 - 0.0015(45) = 0.7825 \]

\[ \beta_1 = 0.97 - 0.0025(45) = 0.8575 \]

\[ C = (0.7825)(45MPa)(0.8575)c(254mm) = 7669c \]

\[ T_s = (64.3mm^2)(693MPa) = 44560 N \]

By setting the compression force equal and opposite to the tension force, \( c = 5.81 \text{ mm} \)

\[ \varepsilon_{sult} = \varepsilon_{ult} \left( \frac{d_s - c}{c} \right) = 0.0035 \left( \frac{71 - 5.81}{5.81} \right) = 0.0393 \quad > \quad \varepsilon_{sy} = 0.0036 \quad \therefore \text{steel has yielded} \]

\[ a = \beta_1 c = (0.8575)(5.81mm) = 4.98mm \]
\[ M_r = A_s f_y \left( d_s - \frac{a}{2} \right) = \left( 64.3 \text{mm}^2 \right) \left( 693 \text{MPa} \right) \left( 71 \text{mm} - \frac{4.98 \text{mm}}{2} \right) \]

\[ Mr = 3052753 \text{ Nmm} = 3.05 \text{ kNm} \]
Flexural Strength of Strengthened Slab-Strip

\[ C = T \]
\[ C = \alpha f'_c \beta_1 cb \]

\[ T_s = A_s f_s \]
\[ T_{frp} = A_{frp} f_{frp} \]
\[ \varepsilon_{frp} = \varepsilon_{ult} \left( \frac{d_{frp} - c}{c} \right) \]

Assume failure is caused by concrete crushing after yielding of the internal steel reinforcement but prior to the debonding or rupture of the FRP strip.

\[ C = (0.7825)(45MPa)(0.8575)c(254mm) = 7669c \]

\[ T_s = (64.3mm^2)(693MPa) = 44560 N \]

\[ T_{frp} = A_{frp} f_{frp} = A_{frp} f_{frp} E_{frp} \varepsilon_{frp} = A_{frp} E_{frp} \varepsilon_{ult} \left( \frac{d_{frp} - c}{c} \right) \]

\[ T_{frp} = (31.2mm^2)(124000MPa)(0.0035) \left( \frac{91mm - c}{c} \right) \]

By setting the compression force equal and opposite to the tension force, \( c = 14.85 \) mm

\[ \varepsilon_{sult} = \varepsilon_{ult} \left( \frac{d_s - c}{c} \right) = 0.0035 \left( \frac{71 - 14.85}{14.85} \right) = 0.0132 > \varepsilon_{sy} = 0.0036 \quad \therefore \text{steel has yielded} \]

\[ \varepsilon_{frp} = \varepsilon_{ult} \left( \frac{d_{frp} - c}{c} \right) = 0.0035 \left( \frac{91mm - 14.85mm}{14.85mm} \right) = 0.0179 > \varepsilon_{ult} = 0.017 \]
\[ \therefore \text{frp has ruptured before the concrete crushed so the initial assumption was wrong} \]

\[ C = T \]

\[ C = \alpha \varepsilon_c \varepsilon_c \beta_1 cb \]

\[ T_s = A_s \varepsilon_s \]

\[ T_{frp} = A_{frp} E_{frp} \varepsilon_{frp} \]

\[ \varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) \]

\[ \therefore C = \alpha \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) \beta_1 cb \]

The \( \alpha \) and \( \beta_1 \) values must be taken from ISIS Design Manual 3 (ISIS 2001)

\[ C = \alpha (30190 \text{MPa}) (0.017) \left( \frac{c}{91 \text{mm} - c} \right) \beta_1 c (254 \text{mm}) = 130360 \alpha \beta_1 \left( \frac{c}{91 \text{mm} - c} \right) \]

\[ T_s = (64.3 \text{mm}^2)(693 \text{MPa}) = 44560 \text{ N} \]

\[ T_{frp} = A_{frp} f_{frp} = A_{frp} E_{frp} \varepsilon_{frp} = 31.2 \text{mm}^2 (124000 \text{MPa}) (0.017) = 65770 \text{ N} \]

Assume \( c = 13 \text{ mm} \), and determine the resultant compressive force using the values from the graphs below:

\[ \varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.017 \left( \frac{13}{91 - 13} \right) = 0.0028 \]
Equivalent stress-block parameter $\alpha$ for concrete strengths of 20 to 60 MPa

\[ \alpha_1 = 0.89 \] \[ \beta_1 = 0.78 \]
Assume \( c = 11 \text{ mm} \), and determine the resultant compressive force using the values from the graphs below:

\[
C = 130360 \alpha_1 \beta_1 \left( \frac{c^2}{91\text{mm} - c} \right) = 130360(0.89)(0.78)\left( \frac{(13\text{mm})^2}{91\text{mm} - 13\text{mm}} \right) = 196074\text{N}
\]

\[
T_s = (64.3\text{mm}^2)(693\text{MPa}) = 44560 \text{ N}
\]

\[
T_{frp} = A_{frp} f_{frp} = A_{frp} E_{frp} \varepsilon_{frp} = 31.2\text{mm}^2(124000\text{MPa})(0.017) = 65770\text{N}
\]

\( C \neq T \therefore \text{another iteration is required} \)

Assume \( c = 11 \text{ mm} \), and determine the resultant compressive force using the values from the graphs below:

\[
\varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.017 \left( \frac{11}{91 - 11} \right) = 0.0023
\]

\( \therefore \alpha_1 = 0.91 \& \beta_1 = 0.72 \)

\[
C = 130360 \alpha_1 \beta_1 \left( \frac{c^2}{91\text{mm} - c} \right) = 130360(0.91)(0.72)\left( \frac{(11\text{mm})^2}{91\text{mm} - 11\text{mm}} \right) = 129185\text{N}
\]

\[
T_s = (64.3\text{mm}^2)(693\text{MPa}) = 44560 \text{ N}
\]

\[
T_{frp} = A_{frp} f_{frp} = A_{frp} E_{frp} \varepsilon_{frp} = 31.2\text{mm}^2(124000\text{MPa})(0.017) = 65770\text{N}
\]

\( C \neq T \therefore \text{another iteration is required} \)

Assume \( c = 10.5 \text{ mm} \), and determine the resultant compressive force using the values from the graphs below:

\[
\varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.017 \left( \frac{10.5}{91 - 10.5} \right) = 0.0022
\]
\[ C = 130360 \alpha_1 \beta_1 \left( \frac{c^2}{91mm - c} \right) = 130360(0.90)(0.72) \left( \frac{(10.5mm)^2}{91mm - 10.5mm} \right) = 115700N \]

\[ T_s = (64.3mm^2)(693MPa) = 44560N \]

\[ T_{frp} = A_{frp}f_{frp} = A_{frp}E_{frp} \varepsilon_{frp} = 31.2mm^2(124000MPa)(0.017) = 65770N \]

\[ C \neq T \therefore \text{ another iteration is required} \]

Assume \( c = 10.3 \text{ mm} \), and determine the resultant compressive force using the values from the graphs below:

\[ \varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.017 \left( \frac{10.3}{91 - 10.3} \right) = 0.00217 \]

\[ \therefore \alpha_1 = 0.90 \& \beta_1 = 0.71 \]

\[ C = 130360 \alpha_1 \beta_1 \left( \frac{c^2}{91mm - c} \right) = 130360(0.90)(0.71) \left( \frac{(10.3mm)^2}{91mm - 10.3mm} \right) = 109510N \]

\[ T_s = (64.3mm^2)(693MPa) = 44560N \]

\[ T_{frp} = A_{frp}f_{frp} = A_{frp}E_{frp} \varepsilon_{frp} = 31.2mm^2(124000MPa)(0.017) = 65770N \]

\[ C \approx T \]

\[ a = \beta_1 c = (0.71)(10.3mm) = 7.31mm \]
\[ M_r = A_s f_y \left( d_s - \frac{a}{2} \right) + A_{frp} E_{frp} \varepsilon_{frp} \left( d_{frp} - \frac{a}{2} \right) \]

\[ M_r = (64.3 \text{mm}^2)(693 \text{MPa}) \left(71 \text{mm} - \frac{7.31 \text{mm}}{2} \right) + (31.2 \text{mm}^2)(124000 \text{MPa})(0.017) \left(91 \text{mm} - \frac{7.31 \text{mm}}{2} \right) \]

\[ M_r = 8.75 \text{ kNm} \]

ACI 440.2R-07 limits the strain in the FRP strip to 70% of ultimate.

\[ \varepsilon_{frp} \leq 0.7 \varepsilon_{frp \text{ ult}} = 0.7(0.017) = 0.0119 \]

\[ \therefore C = \alpha_1 E_c \varepsilon_{frp} \left( c \left( \frac{c}{d_{frp} - c} \right) \beta_1 c \right) \]

The \( \alpha_1 \) and \( \beta_1 \) values must be taken from ISIS Design Manual 3 (ISIS 2001)

\[ C = \alpha_1 (30190 \text{MPa})(0.0119) \left( \frac{c}{91 \text{mm} - c} \right) \beta_1 c (254 \text{mm}) = 91252 \alpha_1 \beta_1 \left( \frac{c^2}{91 \text{mm} - c} \right) \]

\[ T_s = (64.3 \text{mm}^2)(693 \text{MPa}) = 44560 \text{ N} \]

\[ T_{frp} = A_{frp} f_{frp} = A_{frp} E_{frp} \varepsilon_{frp} = 31.2 \text{mm}^2(124000 \text{MPa})(0.0119) = 46040 \text{ N} \]

Assume \( c = 12 \text{ mm} \), and determine the resultant compressive force using the values from the graphs below:

\[ \varepsilon_c = \varepsilon_{frp} \left( \frac{c}{d_{frp} - c} \right) = 0.0119 \left( \frac{12}{91 - 12} \right) = 0.0018 \]

\[ \therefore \alpha_1 = 0.79 \text{ & } \beta_1 = 0.69 \]
\[ C = 91252 \alpha_1 \beta_1 \left( \frac{c^2}{91 \text{mm} - c} \right) = 91252(0.79)(0.69) \left( \frac{(12 \text{mm})^2}{91 \text{mm} - 12 \text{mm}} \right) = 90670N \]

\[ T_s = (64.3 \text{mm}^2)(693 \text{MPa}) = 44560 \text{ N} \]

\[ T_{frp} = A_{frp}f_{frp} = A_{frp}E_{frp}\varepsilon_{frp} = 31.2 \text{mm}^2(124000 \text{MPa})(0.0119) = 46040N \]

\[ C \cong T \]

\[ a = \beta_1 c = (0.69)(12 \text{mm}) = 8.28 \text{mm} \]

\[ M_r = A_s f_y \left( d_s - \frac{a}{2} \right) + A_{frp}E_{frp}\varepsilon_{frp} \left( d_{frp} - \frac{a}{2} \right) \]

\[ M_r = (64.3 \text{mm}^2)(693 \text{MPa}) \left( 71 \text{mm} - \frac{8.28 \text{mm}}{2} \right) \]

\[ + (31.2 \text{mm}^2)(124000 \text{MPa})(0.0119) \left( 91 \text{mm} - \frac{8.28 \text{mm}}{2} \right) \]

\[ M_r = 6.98 \text{kNm} \]
Shear Strength

**ACI 440.1R-06**

\[ V_c = \frac{2}{5} \sqrt{f_c' b_w c} \]

where:

- \( V_c \) = shear resistance provided by the concrete (N)
- \( f_c' \) = concrete compressive strength (MPa)
- \( b_w \) = width of the concrete section (mm)
- \( c \) = cracked transformed section neutral axis depth (mm)

\[ n_{frp} = \frac{E_{frp}}{E_c} = \frac{124000 \text{ MPa}}{31740 \text{ MPa}} = 3.91 \]

\[ n_s = \frac{E_s}{E_c} = \frac{192500 \text{ MPa}}{31740 \text{ MPa}} = 6.06 \]

\[ \therefore A_{frp} = n_{frp} A_{frp} = (3.91)(31.2 \text{ mm}^2) = 122.0 \text{ mm}^2 \]

\[ \therefore A_s = n_s A_s = (6.06)(64.3 \text{ mm}^2) = 389.7 \text{ mm}^2 \]

\[ \Sigma A_i \bar{y}_i = 0 \]

\[ \therefore 254c \left( \frac{c}{2} \right) - 389.7(71 - c) - 122.0(91 - c) = 0 \]
\[ \therefore c = 15.6 \text{ mm} \]

\[ \therefore V_c = \frac{2}{5} \sqrt{f'_c} b_w c = \frac{2}{5} \sqrt{45(254)}(15.6) = 10.6 \text{ kN} \]

ACI 318-05

\[ V_c = 0.17 \sqrt{f'_c} b_w d = 0.17 \sqrt{45(254)}(71) = 20.6 \text{ kN} \]

CSA A23.3-04

\[ V_c = \phi_c \lambda \beta \sqrt{f'_c} b_w d_v \]

\[ d_v = \text{greater of}\ \begin{align*} 
0.9d &= 0.9(71) = 64 \\
0.72h &= 0.72(101.6) = 73 
\end{align*} \]

\[ \therefore d_v = 73 \]

set \( \lambda \) and \( \phi_c \) equal to 1

\[ \beta = 0.21 \ (\text{from } 11.3.6.2(a)) \]

\[ V_c = (1)(1)(0.21)\sqrt{45(254)}(73) = 26.1 \text{ kN} \]