DURABILITY STUDY ON CONCRETE BRIDGE DECKS WITH PULTRUDED FRP STAY-IN-PLACE STRUCTURAL FORMS

by

Raouf Boles

A thesis submitted to the Department of Civil Engineering
in conformity with the requirements for
the degree of Masters of Applied Science

Queen’s University
Kingston, Ontario, Canada
July, 2014

Copyright ©Raouf Boles, 2014
Abstract

This study consists of three phases examining the durability of concrete bridge decks with stay-in-place GFRP structural forms that completely replace the bottom reinforcing bars. Phase I examines the effect of aggressive freeze-thaw (FT) cycles on strength of small scale decks. The concern has been whether entrapped moisture may cause ‘frost-jacking’ of the form. Eleven specimens were built, each using two spliced flat GFRP plates with T-shape ribs, spanning the gap between girders. The study simulated various surface treatments of the form as well as unbonded and bonded lap splices. The decks were cracked before being saturated and subjected to up to 300 FT cycles at +5°C to -18°C core temperatures. Some specimens were thawed without being submerged and one specimen had perforated forms for drainage. Subsequent testing to failure showed no reduction in ultimate capacity or stiffness, despite the 23% reduction in tensile strength of GFRP coupons from the same form, because failure was governed by punching shear.

Phase II compares the GFRP form tested in Phase I to another corrugated form, using short one way slabs to trigger a shear-bond failure. Nine slabs with different surface treatments were fabricated and some were exposed to the same FT cycles. It was clearly shown that flat-ribbed forms are superior to corrugated ones, as no loss in strength occurred after FT exposure, whereas corrugated form-specimens lost 18-21%. This is attributed to the anchorage advantage provided by the T-shape rib embedment in concrete.

In Phase III accelerated aging of the two GFRP forms is studied in 3% salt solution at 23, 40 and 55°C for up to 224 days, using 170 coupons to establish tensile
strength retentions. Data were assessed using Analysis of Variance (ANOVA). It was shown that the tensile strength retentions of both forms were similar and reduced from 77 to 63% as the temperature increased from 23 to 55°C. Results also showed that the polymer matrix is not fully degraded by the hydrolysis as no significant changes occurred in glass transition temperature. When data was fitted in the Arrhenius service life model, it showed that after 100 years, the ribbed form will suffer more deterioration than the corrugated one as the strength retentions at a location with annual mean temperatures of 10°C were 42 and 61%, respectively.
Acknowledgements

I would like to thank Dr. Amir Fam for giving me this opportunity to be one of his students. In particular, for all his support and guidance during the past two years under his supervision towards my master’s degree. Also, my sincere thanks to the technical staff at Queen’s University, especially Maxine Wilson, Neil Porter, Lloyd Rhymer, Jaime Escobar, Paul Thrasher and Adam Reczek for all their technical support throughout my master’s. I would also like to express my appreciation to all my colleagues at Ellis Hall especially, Mark Nelson, Douglas Tomlinson, Emir Beriker, Kenneth Mak, Gergely Mucsi and Ramy Awad for all their hands-on contribution over the past two years.

I also wish to acknowledge the financial support provided by the Ministry of Transportation Ontario (MTO).

Finally but most importantly, I would like to thank my lovely wife Nermeen for all her unwavering support and encouragement during this journey.
Table of Content

Abstract .................................................................................................................................ii

Acknowledgements .............................................................................................................iv

List of Figures .......................................................................................................................viii

List of Tables .........................................................................................................................xi

Chapter 1: Introduction .......................................................................................................1

1.1 General .............................................................................................................................1

1.2 Scope................................................................................................................................2

1.3 Objectives ........................................................................................................................3

1.4 Background .......................................................................................................................3

1.5 Thesis Format ....................................................................................................................9

Chapter 2: Durability of Bridge Deck with FRP Stay-in-Place Structural Forms under
Freeze-Thaw Cycles .............................................................................................................10

2.1 Introduction ......................................................................................................................11

2.2 Experimental Program ....................................................................................................13

2.2.1 Test Specimens ..........................................................................................................13

2.2.2 Test Parameters ..........................................................................................................15

2.2.3 Materials ......................................................................................................................17

2.2.4 Fabrication of Specimens ...........................................................................................19

2.2.5 Specimen Cracking before Conditioning ....................................................................19

2.2.6 Test Setups ..................................................................................................................21

2.3 Experimental Results ....................................................................................................23

2.3.1 Effect of Surface Treatment of FRP Forms .................................................................23
2.3.2 Effect of Freeze-Thaw Cycles ......................................................... 24
2.3.3 Effect of Submersion and Form Perforation for Drainage ............... 25
2.3.4 Failure Mode ............................................................................. 25
2.3.5 GFRP and Concrete Strains ....................................................... 26

2.4 Summary ..................................................................................... 27

Chapter 3: The effect of the shape of FRP Stay-in-Place Structural Forms on Durability of Concrete Members under Freeze-Thaw Cycles ......................................................... 37

3.1 Introduction ................................................................................... 38

3.2 Experimental Program ............................................................. 39

3.2.1 Test Specimens and Parameters ................................................ 39
3.2.2 Materials .................................................................................. 42
3.2.3 Fabrication of Test Specimens .................................................... 43
3.2.4 Environmental Chamber Setup and Instrumentation .................... 44
3.2.5 Structural Testing Setup and Instrumentation ............................... 44

3.3 Experimental Results .............................................................. 45

3.3.1 Effect of Surface Treatment of GFRP Forms in Control Specimens .... 46
3.3.2 Effect of Freeze-Thaw (FT) Cycles ............................................ 48

3.4 Summary ..................................................................................... 50

Chapter 4: Durability Study on Pultruded FRP sections used in Bridge Deck Applications ......................................................................................... 59

4.1 Introduction .................................................................................. 59

4.2 Experimental Program ............................................................. 60

4.2.1 Materials .................................................................................. 61
4.2.2 Test Specimens and Parameters .................................................. 62
4.2.3 Fabrication of Coupons .............................................................. 63
4.2.4 Test Setups, Procedures and Instrumentation .............................. 63

4.3 Experimental Results ................................................................. 65
4.3.1 Results of Control Tension Specimens .................................... 66
4.3.2 Results of Environmentally Aged Tension Specimens ................ 67
4.3.3 Results of Microstructure Analysis Tests .................................... 69
4.3.3 Statistical Assessment Using Analysis of Variance ..................... 71

4.4 Summary ....................................................................................... 73

Chapter 5: Conclusions ..................................................................... 88
5.1 Durability of Bridge Deck with FRP Stay-in-Place Structural Forms under Freeze-Thaw Cycles .......................................................... 88
5.2 The Effect of the Shape of FRP Stay-in-Place Structural Form on Durability of Concrete Members under Freeze-Thaw Cycles ..................... 89

5.3 Durability Study on Pultruded FRP sections used in Bridge Deck

References ......................................................................................... 93
List of Figures

Figure 2-1 Schematic diagram of test specimens ................................................................. 29
Figure 2-2 GFRP SIP Stress-Strain Curves ................................................................. 29
Figure 2-3 Variation of concrete strength with time...................................................... 30
Figure 2-4 Surface preparations of FRP SIP form.......................................................... 30
Figure 2-5 Environmental exposure arrangements and protocol ..................................... 31
Figure 2-6 Test Setup ........................................................................................................ 31
Figure 2-7 Load – deflection responses of specimens with and without FT exposure .... 32
Figure 2-8 Deflected shapes of specimens S1, S2 and S3 in both directions ................. 32
Figure 2-9 Load – deflection responses of specimens without surface treatment after various Freeze-Thaw cycles ..................................................................................... 33
Figure 2-10 Variation of deck strength with Freeze-Thaw cycles with different configurations .................................................................................................................. 33
Figure 2-11 Load-deflection responses of specimens with and without drainage holes compared to submerged specimen after 300 FT cycles ........................................... 34
Figure 2-12 Punching shear failure, top view .................................................................. 34
Figure 2-13 Slip at GFRP form splice in decks with various surface preparations ....... 35
Figure 2-14 Load- strain responses in FRP SIP forms at various locations ................. 36
Figure 2-15 Load- top strains responses at different locations ........................................ 37
Figure 3-1: Schematic diagram of test specimens, setup and instrumentation .......... 52
Figure 3-2: GFRP SIP form stress-strain curves ............................................................. ..53
Figure 3-3: Various surface preparations ....................................................................... 53
Figure 3-4: Conditioning setup showing the water tank inside cold room..................54
Figure 3-5: Test setup .................................................................54
Figure 3-6: Load – deflection responses ............................................55
Figure 3-7: Load – FRP bottom strain responses..................................55
Figure 3-8: Load – concrete top surface strain responses.........................56
Figure 3-9: Load – slip responses......................................................56
Figure 3-10: Failure modes..............................................................57
Figure 4-1: Various types of pultruded GFRP sections used in bridge decks..........78
Figure 4-2: Schematic view of coupons cut in the longitudinal direction from the GFRP panels ........................................................................................................78
Figure 4-3: Environmental tank setup.........................................................79
Figure 4-4: Stress-strain curves for R-GFRP and C-GFRP for: .........................80
Figure 4-5: Tensile strength......................................................................81
Figure 4-6: Tensile strength retention for R-GFRP and C-GFRP over 224 days at various temperatures ........................................................................................................81
Figure 4-7: Failure modes........................................................................82
Figure 4-8: Elastic moduli ..........................................................................82
Figure 4-9: Micrographs of the external surface...........................................83
Figure 4-10: Micrographs at the fibre/matrix interface before mechanical tests......84
Figure 4-11: Variation of tensile strength retention with time logarithmic ..............85
Figure 4-12: Arrhenius plots for: a) R-GFRP and b) C-GFRP ..........................86
Figure 4-13: Predicted relationship between the retained tensile strength and service life at annual mean temperatures of 3°, 10°, and 20°C
List of Tables

Table 2-1: Summary of test matrix .................................................................................. 28
Table 3-1: Summary of test matrix .................................................................................. 51
Table 3-2: Summary of test results .................................................................................. 51
Table 4-1: Test Matrix .................................................................................................. 75
Table 4-2: Summary of test results .................................................................................. 76
Table 4-3: Results of ANOVA statistical analysis ......................................................... 77
Chapter 1: Introduction

1.1 GENERAL
Over the past several decades, deterioration of infrastructure has been widely recognized by scholars throughout cold climate regions, particularly in North America, as a serious and progressing problem. In these regions, especially Canada, concrete bridges are exposed to harsh climate conditions, namely freeze-thaw cycles along with dry-wet conditions. In winter, using the de-icing salt on roads accelerates the chloride attack of the internal steel reinforcement. As a result, this problem has drawn more attention from scholars to investigate a new material that has corrosion resistance and a major focus was placed on fibre reinforced polymers (FRPs). These materials are produced from fibres, which give the strength, and resin, which holds and distributes the load equally to the fibres. These materials are versatile in that it are produced in various forms, including FRP shapes and panels, bars, sheets, and laminates. This makes these materials suitable for new construction and retrofitting existing bridges and structures. FRPs have been commonly used in several applications related to concrete structures due to their advantages that outweigh their limitations (Bakis et al. 2002). High strength, light weight and corrosion resistance are the primary advantages of FRPs compared to conventional reinforcements. Also, developing a new approach for construction in order to reduce the labour cost as well as the construction time would also result in reducing the overall bridge cost. Hence, using FRP Stay-in-Place (SIP) formwork system for bridge decks has combined advantages based on both FRPs’ durability and constructability (Berg et al.
2006). Generally, FRP SIP forms act as a bottom reinforcement layer instead of using the conventional steel bars.

Significant research has been carried out at Queen’s University in the past eight years to investigate the performance of bridge decks reinforced by FRP SIP panels using different panels shapes with different bond systems between the concrete and the FRP panels (Fam and Nelson, 2012; Nelson and Fam, 2012; Nelson et al. 2013; Richardson et al. 2013; Nelson and Fam, 2014) However, these studies focused on static and fatigue tests at room temperatures.

1.2 SCOPE
This thesis addresses the effect of laboratory-simulated environmental conditions on the durability and structural capacity of decks with FRP SIP forms. It includes three phases. The first and second phases of this research investigate the effect of 300 freeze-thaw cycles on the decks with various bond mechanisms at the interface between the panel and concrete. The first phase included 11 small-scale bridge decks reinforced by ribbed FRP panels. Three decks were cast and stored at room temperature, while the other specimens were exposed to 100, 200, and 300 freeze-thaw cycles. The second phase investigated nine slabs reinforced by either ribbed or corrugated FRP panels to study the effect of form shape on freeze-thaw performance. The third phase focuses on the effect of a saltwater solution, which represents de-icing salts, on (both the ribbed and corrugated FRP panels) used in the second phase. Eighty coupons were cut from each type, and made into three groups exposed to three different temperatures and then tested in tension at five time intervals and compared to control samples. The samples were immersed in
three baths of 3% saltwater at 23°C, 40°C, and 55°C. Sets of five coupons was removed from the tanks after 14, 33, 97, 150, and 224 days and tested in tension. The results of the aged coupons tests were compared with the control samples to observe the effects on the elastic modulus and ultimate tensile strength retention of the GFRPs. The study also predicts the strength retention of the GFRPs over their service life by applying the Arrhenius model to the laboratory results.

1.3 OBJECTIVES

The objectives of this research work are as follows:

1. To investigate the effect of freeze-thaw cycles on the performance of bridge decks with structural FRP stay in place forms.

2. To investigate the effect of the shape and geometric configuration of the FRP forms on the slab performance under freeze-thaw effects.

3. To assess various bond mechanisms at the interface between concrete and FRP, including adhesive coating and coarse aggregate versus no treatments at all, before and after freeze-thaw cycles.

4. To investigate the accelerated aging effect of the GFRP panels using salt solution at various elevated temperatures.

5. To predict the tensile strength retention of the GFRP forms over their service life using the Arrhenius model.
1.4 BACKGROUND

Limited research has been conducted in terms of durability and environmental effects on concrete bridge decks with FRP SIP forms, compared to the large volume of work on durability of FRPs used as rebar or sheets and laminates for retrofitting purposes. In this literature review, a summary is provided of some previous studies that were carried out on FRP materials and RC concrete beams strengthened with FRP laminates exposed to different environmental conditions.

Baumert et al. (1996) conducted tests to investigate the effect of low temperatures on FRP-plated concrete beams. They concluded that, no significant damage occurred after FRP beams were subjected to those cycles. One of the tests that was carried out by Green et al. (1997) on RC concrete beams strengthened with CFRP plates after being subjected to 50 cycles from +15 to -18°C arrived at similar conclusions. Del Mar Lopez et al. (1999) conducted a number of tests on small scale FRP-plated concrete beams after being subjected to 300 freeze-thaw cycles, and found significant reduction in both capacity and maximum deflection.

To investigate the bond performance between FRP and concrete beams after exposure to freeze-thaw cycles, (Green et al. 2003) tested beams strengthened using CFRP or GFRP sheets under 200 freeze-thaw cycles. Insignificant deterioration in capacity was observed after freeze-thaw cycles. Green et al. (2000) studied the effect of 300 freeze-thaw cycles on the bond between FRP plates and concrete using uniaxial CFRP strips bonded to the beams along with single lap pull-off specimens. Each cycle
consisted of 16 hours of freezing in air and 8 hours of thawing in water. They concluded that there was no reduction in load capacity and also that the bond was not significantly damaged.

Yun and Wu (2011) conducted single face shear tests on CFRP bonded to concrete specimens in order to investigate bond performance. They indicated that, bond length and bond stiffness between FRP-concrete reduced with an increasing the number of freeze-thaw cycles. Karbhari and Engineer (1996) also conducted freeze-thaw tests on FRP sheet bonded to small cement mortar beams using a wet layup technique. It was determined that immersion in water resulted in about 10% reduction in the slab capacity since the glass transition temperature of the FRP resin dropped significantly. Tuakta and Buyukozturk (2011) conducted an experimental program to investigate the degradation of the bond between FRP strips and concrete using an adhesive layer. The findings from this study showed a significant reduction in bond strength after exposure to wet-dry cycles. Karbhari (1997) also performed a series of tests related to short-term durability of the bond at the composite-concrete interface. The results showed that some deterioration occurred in composites as well as at concrete-composite interface due to entrapped moisture that leads to significant degradation at the fibre-matrix interface. Major research has been carried out by Nkurunziza (2005) on concrete beams with bonded FRP sheets, to address the behaviour of the beams after exposure to many freeze-thaw cycles. Neville (2005) concluded that if an air entrainment is used within concrete mix, there is no effect of freeze-thaw cycles on the beam capacity.
During the freezing period, the concrete pore solution in concrete expands causing internal hydraulic pressures and therefore frost damage occurs (Cai 1998). Under microscopic observation, the nature of micro cracks between the mortar and aggregate in concrete significantly changes (Soroushian and Elzafraney, 2004). In particular, freeze-thaw causes propagation and joining of these micro cracks at the interface between the aggregate and paste.

The combined effect of immersing specimens in chloride salt solution during thawing is more severe than using water only (Sun et al. 2002). Davalos et al. (2008) performed tests to evaluate the durability of the interface between the bonded sheet and concrete. The specimens were subjected to freeze-thaw cycles saturated in calcium chloride that represent the de-icing salt used in winter on bridges. They concluded that, the differential movement between the concrete and GFRP and crack propagation at the interface increased with increasing number of cycles. It should be noted that, the possible failure modes for simply supported beam included: rupture of the FRP composite, interfacial failure between the adhesive and composite or concrete substrate, cohesive failure in the adhesive, alternating crack path between the two interfaces, shear failure or flexural failure as reported by Buyukozturk et al. (2004). However, one mode of failure was recorded when Colombi et al. (2010) assessed the bond strength under freeze-thaw cycles. The pull-off debonding tests were carried out on concrete blocks with bonded CFRP strips for an exposure period of up to 200 cycles. All specimens failed in debonding that mainly occurred within concrete substrate and not in the adhesive layer.
To investigate the effects of freeze-thaw cycles on FRP materials only, research was conducted by Dutta (1988) on glass-epoxy FRPs subjected to 150 freeze-thaw cycles from +23 to -40°C. The study reported 10% reduction in their tensile strength. Similarly, several tests were carried out on GFRP bars after exposure to different environmental conditions like sea water, dry/wet cycles, alkaline solutions, and ordinary tap water for 6, 12, and 18 months (Al-Salloum et al. 2013). It was found that some reduction in FRP tensile strength has been found after being subjected to those cycles. A series of durability tests on glass-fibre reinforced polypropylene composite laminate were conducted (Robert et al. 2010) after immersing in salt water of 3% concentration or tap water. The results showed that the salt solution leads to degradation of the interface between the fibre and matrix. In addition, the immersion in tap water had less influence than salt-water solution on the flexural properties of the samples after 168 days. Similarly, Chen et al. (2007) conducted durability tests on GFRP bars after immersion in various solutions namely: water, alkaline solution and saline solution at 20°C, 40°C and 60°C. It was determined that, the higher temperature caused more degradation to the GFRP bar tensile strength over a period of 240 days. As a result, it was concluded that increasing temperature will induce more stress to the fibre and result in decreasing the GFRP bar strength (Vijay and GangaRao, 1999).

Nkurunziza (2005) performed durability tests on GFRP rebar to investigate their short term durability after exposure to alkaline solution, high temperatures, and sustained loads, concurrently. The results of this research showed that a significant degradation in the tensile strength occurs and depends on the immersion period. It should be noted that,
the composite materials absorbed moisture and experienced high temperature-induced stress not only to the fibre and matrix but also to the interface. As a result, the tensile strength gradually reduced over time.

Extensive research was conducted on coupons of glass-reinforced epoxy to determine the degradation after exposure to various solutions (Kajorncheappunngam et al. 2002). The aged samples were exposed for up to 5 months in various liquid medias separately: distilled water, saltwater, alkaline solution, and acid solution at 23° and 60°C. The results revealed that, the alkaline and acid solutions reduced the FRP strength by up to 70% at room temperature. Similarly, an experimental program was conducted on three FRP systems namely GFRP fabrics, CFRP fabrics, and CFRP plates to assess their durability (Cromwell et al. 2011). The coupons were immersed in various solutions including; water, saltwater, alkaline, dry heat, diesel fuel, weathering effects, and freeze–heat exposure. After completion of the exposure period, the samples were tested in tension, shear, and bond according to ASCE guidelines. The results showed that, the glass transition temperature \( T_g \) of the material may be approached at 60°C and therefore, the properties of the material would be markedly changed. Also, the results showed that, the most appropriate method to assess the extreme environmental conditions of FRPs is the exposure to saltwater as well as alkaline solutions. Also, the absorbed water at the interface between the matrix and the fibres induced microcracks to the composite so the flexural and tensile strength of the fibres gradually decreased. Moreover, the absorbed moisture would reduce the glass transition temperature \( T_g \) of the polymers, which represents the phase of material change from the glassy to the rubbery
phase (Kumar and Gupta, 1998). The reduction in $T_g$ resulted in physical change and softened the polymers, promoting more creep deformations upon drying. If the wet-layup process was used to fabricate the GFRP sheets at room temperature, moisture condition would significantly reduce the $T_g$ of the polymers as the resin is not fully cured. In addition, the absorbed water can lead to physical and chemical permanent changes within the fibres themselves or between the fibres and matrix, as well. As the volume of the sample increase due to water ingress, the adhesion between the fibres and matrix would be gradually lost. Microcracks at the fibres-composite interface affect the bond and degrade the stiffness and strength of the composite as reported by (Schutte, 1994).

In summary, all the mentioned research studies concluded that there would be an aggressive effect of freeze-thaw cycles on the FRP materials because of loss of adhesion between the fibre and matrix. Nevertheless, they are not in agreement with the effect of this aggressive exposure to the bond between the FRP bonded sheet and the concrete beam.

1.5 THESIS FORMAT
This thesis is composed of five chapters, which contain the introduction, three chapters in manuscript format, and a conclusions chapter. Chapter 1 provides the main ideas of research work, objective, and scope of the project as well as a background literature review. Chapter 2 describes the first phase of this research project, which includes an investigation of freeze-thaw effect on the performance of small-scale bridge decks made using FRP SIP forms with complete detailing of connections and boundary conditions, examining various bond mechanisms. Chapter 3 includes the second phase of this thesis,
which aims to investigate the effect of freeze-thaw cycles on one-way slabs, comparing two types of FRP forms, namely ribbed and corrugated panels, using different kinds of bond at the interface. Chapter 4 presents a durability study of environmental effect of salt solution and elevated temperatures on the GFRP ribbed and corrugated panels used in the first and second phases. Chapter 5 includes the conclusions of this research
2.1 INTRODUCTION

Bridge decks can benefit tremendously from the concept of FRP SIP structural forms for two reasons: speed of construction because the forms are permanent, and the corrosion-resistance of FRP in presence of de-icing salts on the road. Recently, a state-of-the-art article that provides a broad perspective on this technology, specifically for bridge decks, was published (Nelson et al. 2013). It addresses advantages and limitations of the system, various configurations of the SIP forms, detailing of panel-to-girder connection, construction procedures, cost effectiveness and research needs. With regard to barriers to further adoption of this system in practice, the article identified durability under Freeze-Thaw (FT) cycles as a key research need. Concerns have been raised with regard to ingress of moisture and possible frost-jacking between the FRP form and concrete. Despite the barriers to widespread applications, several field applications have been reported in the literature, including the Salem Avenue Bridge in Ohio (Reising et al. 2004), Route US-151 Bridge in Wisconsin (Berg et al. 2005), Greene County Bridge in Missouri (Matta et al. 2006), and the Black River Falls Bridge in Wisconsin (Oliva et al. 2007).

Several research studies have been conducted on bridge decks with FRP SIP forms of various configurations. This includes decks with thin plate bonded to the bottom

---

1 This Chapter has been submitted for publication as the following journal paper: Boles, R. and Nelson, M. and Fam, A. (2014) “Durability of Bridge Deck with FRP Stay-in-Place Structural Forms under Freeze-Thaw Cycles.”, ASCE Journal of Composites for Construction, Under review

2 Nelson, M. contributed by assisting during initial design and fabrication stages
of a layer of grid reinforcement (Ringelstetter et al. 2006), corrugated plates with pin-and-eye interlocking joints (Fam and Nelson, 2012), and flat plates with T-shape ribs (Nelson and Fam, 2013). The studies addressed the effect of boundary conditions and the effect of deck width when testing discrete deck segments, compared to a full bridge deck, as well as the effect of interface bond (Nelson et al. 2013 and Nelson and Fam, 2014), splices of FRP forms (Nelson et al. 2014).

Traditionally, the effect of FT cycles on concrete in cold regions can be mitigated by the use of air entrainment, but FT cycles also have a negative impact on FRP composites as was first reported by Dutta (1988). GFRP laminate subjected to 150 FT cycles at +23°C to -40°C showed a 10% reduction in tensile strength. For externally bonded Carbon-FRP plates used for rehabilitation of concrete, Green et al. (2000) investigated the effect of up to 300 FT cycles at +15 to –18 °C on the bond. The study showed that bond was not significantly damaged.

This chapter aims at filling an important gap with regard to the technology of bridge decks with FRP SIP forms, particularly looking at their durability and the impact of FT cycles on their strength and integrity. Factors that are thought to impact performance under FT exposure have been considered in the study. This includes pre-cracking specimens before exposure, various surface treatments and bond conditions at the form-concrete interface, bonded versus unbonded lap splice of the forms, non-perforated versus perforated FRP forms for drainage, and submerged versus non-submerged deck specimens during thawing by water.
2.2 EXPERIMENTAL PROGRAM
The following sections provide details of the experimental program including test specimens, parameters, materials, fabrication, specimens’ cracking before conditioning, test setups, and instrumentation

2.2.1 Test Specimens

*Background of the design:* Nelson and Fam (2014) tested a complete bridge system with GFRP SIP forms for the concrete deck. The bridge was constructed at 1:2.75 scale with special attention to all detailing, including connection of deck to girders, continuity of the deck in the transverse direction over several girders, and using end diaphragms. The study aimed at stimulating as close as possible accurate boundary conditions found in a full scale bridge and estimating the equivalent service load at this scale. For a full scale bridge, the service load is typically considered to be the half-axle load plus maximum dynamic allowance, which is 122.5 kN, of the CL-625 design truck of CSA S6-06 used in Canada. Analysis of the experimental results of that study showed that the equivalent service load at this particular scale is 24.3 kN.

It is not realistic, however, to always build complete bridges (i.e. with multiple full-length girders), as that in the study mentioned above, when testing various aspects of bridge decks such as the durability investigation reported in this chapter. Therefore, researchers typically test discrete sections of bridge decks with adequate width. Nelson et al. (2013) tested several discrete bridge deck segments spanning two girders, representing portions of the full bridge mentioned above, with exactly the same design and materials but with various widths. It was concluded that a width in the direction parallel to girders (i.e. traffic direction) equal to at least 1.33 times the girder spacing is
sufficient to reach the same ultimate load of the full bridge and also the same punching shear failure mode. This design is also provides two deck overhangs on either side of the two supporting girders to accommodate the development length of the top layer of GFRP rebar for the negative moment regions above the girders. Nelson et al. (2014) then refined the test setup further by introducing a reusable support system that can be bolted to the deck specimens and provide equivalent boundary conditions to real conditions, to allow for reusing the setup for several specimens. This setup was convenient for the current study as the concrete decks can be cast and conditioned easily without the elaborate and heavy support system, which is only needed when loading the deck.

Description of test specimens: The 1220x1000x65 mm specimens in this study were also designed and built at 1:2.75 scale, similar to the previous studies. Figure 2-1 shows the general layout of a typical deck specimen, including cross-section views. The specimens have a girder spacing of 665 mm, equivalent to 1829 mm (6 ft. nominal) at full scale (Fig. 2-1(b)). The 1000 mm width in the direction parallel to girders (and traffic) (Fig. 2-1(a)), was selected to be 1.5 times girder spacing, which exceeds the 1.33 minimum ratio recommended by Nelson et al. (2013). The deck included a 278 mm overhang from each side beyond the center of the support girders, in order to accommodate the development length of the top GFRP rebar (Fig. 2-1(b)). The scaled total deck thickness was 65 mm. The width of the concrete component of support girder is 140 mm (scaled from AASHTO Type III precast girders at full scale) and included U-shape Ø6@125 mm protruded steel stirrups embedded in the deck. The GFRP SIP form was a flat plate with T-shape ribs. The plates spanned the gap between girders such that the ribs were in the direction
normal to traffic (Fig. 2-1(c)). As the plates have a limited width, two plates had to be spliced and the splice was located at mid-width, directly under the load (Fig. 2-1(a)). At the splice, the plates were overlapped 18 mm, scaled from the 50 mm at a full scale design (Nelson and Fam, 2013). At the supporting girders, the GFRP panels extended 30 mm into the supports to simulate the 75 mm at full scale. The GFRP panels completely replaced the bottom layer of rebar. For the top reinforcement, 6.35 mm GFRP bar orthogonal mesh with 125 mm spacing was designed and used according to CSA S6-06.

2.2.2 Test Parameters
Table 2-1 provides a summary of the test matrix. Eleven deck specimens were tested in this study; including three control specimens S1-S3 tested to failure without FT exposure, and eight specimens S4-S10 exposed to FT cycles before testing to failure. The main parameters included in this study are:

a) Panel surface treatment: the bond and interface condition between the concrete and form is a key consideration in this system. Three surface preparations were considered for the GFRP SIP form, namely: (i) just surface cleaning, referred to in Table 2-1 as “Nothing” where concrete is cast directly on the forms as received (specimens S3 and S6-S10), (ii) using a special adhesive coating that bonds freshly-cast concrete to GFRP form (specimens S1, S4-a and S4-b), or (iii) applying a layer of coarse aggregates adhesively bonded to the forms to create a rough texture before concrete casting (specimens S2 and S5).

b) Splice treatment: two treatments were applied at GFRP plates overlap of the splice, namely: (i) nothing, meaning one plate simply rests on the other (specimens S3 and S6-S10, which also have no surface treatment of the panels, thereby representing the
simplest possible construction procedure), or (ii) applying both adhesive and mechanical bond using 4.59 mm steel screws spaced at 33 mm, as designed by Nelson et al (2014) (specimens S1, S2, S4, S5, which also have surface treatments for the panels). Specimens S1-S3 can then be compared to assess surface treatment of plates and splice without FT, while specimens S4, S5 and S8 can be compared to assess both parameters after maximum FT exposure.

c) **Number of FT cycles:** three levels of FT exposure intensities were implemented, namely 100, 200 and 300 cycles (specimens S6-S8) to assess the progression of damage, if any, over time, compared to unexposed specimen S3. This parameter was studied for the specimens without any surface treatment at interface, thought to represent the worst case scenario of bond.

d) **Submersion during thawing:** in this study, freezing is done in air while thawing is done by water, where the tank is flooded and the specimens become fully submerged. This was considered a more severe condition compared to real life with regard to moisture intake. To assess this effect, specimen S9, which is identical to S8, was placed on a special setup within the tank such that thawing is by water that flows over the surface and then falls off without the specimen being submerged in the tank.

e) **Drainage of entrapped moisture:** it is hypothesized that under FT exposure, moisture might get trapped at the SIP form-concrete interface causing increase in concrete volume during freezing, potentially causing ‘frost-jacking’ that separates the form from concrete. To examine this effect, specimen S10 was identical to S9, except that the SIP form was perforated by drilling 5 mm diameter holes at 100 mm spacing in
both directions before casting, to provide path for water egress through the soffit of the slab. Similar to S9, S10 also was not submerged during thawing.

### 2.2.3 Materials

**GFRP SIP panels:** Commercially available pultruded GFRP panels were used in this study as SIP formwork. The 4.2 mm thick flat plate had T-shaped ribs of the same thickness, spaced at 100 mm. Twenty coupons were cut from the plate in both directions and tested in tension according to ASTM D3039/D3039M-08. Ten of the coupons were placed in the FT tank with the deck specimens, under the same conditions for the full duration, and then tested in tension after exposure to 300 FT cycles. Figure 2-2 shows the stress-strain curves in the longitudinal (parallel to ribs) and transverse directions. Given that the primary direction of continuous fibres is longitudinal, the response is quite linear in this direction with tensile strength and modulus of 368 MPa and 32.5 GPa, while in the transverse direction is slightly nonlinear, with tensile strength and modulus of 29.8 MPa and 8.2 GPa, respectively.

Figure 2-2 also shows that the 300 FT cycles had a pronounced effect on the coupons. The average tensile strength and modulus in the longitudinal and transverse directions decreased to 278.5 MPa, 28.8 GPa, 23.2 MPa and 8.0 GPa, respectively. This represents about 23% reduction in strength and 11% reduction in modulus.
GFRP rebar: V-Rod #2 GFRP rebar mats of 6.35 mm diameter were installed in all specimens as top reinforcement. The tensile strength and elastic modulus provided by the manufacturer were 784 MPa and 46.1 GPa, respectively.

Concrete: Ready-mix concrete with 10 mm pea stone aggregate, 7% air entrainment and 200 mm slump was used. Forty 102x204 mm cylinders were cast and some were subjected to the same freeze-thaw cycles as the decks before being tested in compression according to the ASTM C39 (2010) at a similar time to the deck testing. Figure 2-3 shows the variation of compressive strength with time. For the first six months, at room temperature, the strength reached 24.8 MPa, that is when the control deck specimens were tested. Around the same time also, the conditioning under FT cycles started for the remaining specimens and associated cylinders. After 200 and 300 FT cycles (i.e. after additional 67 and 100 days, respectively, in the tank) the measured strength was 23.4 and 27.3 MPa, respectively. Two aspects occurred simultaneously:

(i) Concrete curing: the longer the concrete is cured during thawing cycles, the higher strength is gained.

(ii) Concrete leaching: during curing the concrete leached and thus it would loose some of its chemical constituents such as calcium.

Consequently, it was observed that the loss and gain in concrete strength was primarily dominant by the two aspects mentioned above and not by the FT cycles.

Steel bars and stirrups: 2#10M steel bars were provided in the concrete support girders. Tension tests gave a 435 MPa yield strength. U-shape 6 mm diameter stirrups were also provided in the support girders, spaced at 125 mm. Tension tests gave a yield strength and modulus of 462 MPa and 195 GPa, respectively.
**Epoxy Adhesives:** Two types of adhesives were used. When bonding the FRP panel-to-panel lap splice and when bonding 4-9 mm diameter silica stones to the surface of the FRP panel, Sikadur 30 was used. It is a high modulus, high strength, and high viscosity epoxy paste adhesive with a 20.6 MPa bond strength as reported by the manufacturer. For bonding freshly cast concrete to the FRP panels, Sikadur 32 Hi-Mod, a high modulus, and low viscosity epoxy adhesive with manufacturer reported bond strength of 13.1 MPa was used to coat the surface with a thin layer, shortly before concrete casting.

### 2.2.4 Fabrication of Specimens
As shown in Fig. 2-1(a), each specimen included two GFRP panels, side-by-side. The panels were received in a standard 518x6100 mm size. They were then cut to 585 mm lengths in the direction parallel to the ribs. This length was equal to the 525 mm clear span between girders plus 30 mm on either side, where the form rests on the girder. The panel width was also trimmed to 509 mm, to achieve the 1000 mm design width of the specimen, with the 18 mm overlap between the two panels (Fig. 2-1(a, c)). Wooden forms were built for the concrete beams and overhang sections. The steel cages of the beams were placed in position and vertical PVC pipes were placed in the beams to facilitate anchoring the deck to the test setup as will be discussed later. The GFRP SIP forms were placed on the forms of the beams with 30 mm embedment on both sides, and the top rebar mesh was secured in position before casting. It should be noted that in reality the girders are precast with a roughened surface of the flange and also include protruding stirrups to bond to the cast-in-situ deck. Nelson and Fam (2013) built specimens using this field practice and noted that no relative displacement occurs.
between deck and girders throughout the test. As such, in the current study, the deck and support beams were cast monolithically (i.e. in one cast for simplicity) with the stirrups of the beam also being embedded in the deck as in practice.

Figure 2-4(a) shows the GFRP forms without any surface preparations or splice bond (specimens S3, S6-S9). For specimens S1, S2, S4 and S5, the splice overlap was first adhesively bonded, and then 4.59 mm mechanical screws were installed at 33 mm spacing (Figs. 2-4(b and c)) after adhesive cure as per the recommendation of Nelson et al (2014). Figure 2-4(b) also shows the application of a wet adhesive coating to the horizontal surfaces of the panels just before casting concrete in specimens S1 and S4. Figure 2-4(c) shows the bonded coarse aggregates to the panels of specimens S2 and S5, which was applied to the plate only and not to the T-shape ribs. Figure 2-4(d) shows the grid of drilled drainage holes in the panel of specimen S10.

2.2.5 Specimen Cracking before Conditioning
As bridge decks are typically cracked in real life and since this could aggravate the FT effect by maximizing the moisture intake, it was decided to provide controlled cracking before exposure. Each specimen was installed and connected to the setup and subjected to three loading cycles, up to 35 kN, to insure that cracks formed and opened and closed twice. This load was calculated as 1.5 times the service load of 24.3 kN and ensured that all specimens, regardless of bond conditions at splice or interface, have cracked as was visibly evident on the top surface in the negative moment regions above the girders. Load-deflection data for this phase of testing were collected. After the three cycles, the decks were released from the loading setup and were ready for environmental exposure.
2.2.6 Test Setups

Environmental chamber setup and instrumentation: A large wooden tank was built to keep the concrete slabs submerged in water during FT cycles and was placed inside an environmental chamber (Fig. 2-5(a)). A rubber liner was placed on the inner surface to prevent water leakage. The tank was fitted with a drain at the lowest point and an overflow outlet near the top. Specimens S4-S8 were stacked in two layers in the tank (Fig. 2-5(b)) such that during the thawing period, when the tank is filled with water, they are completely submerged. A special arrangement was made for specimens S9 and S10, which were placed in an elevated position above the maximum water level in the tank, as they simulate real life conditions where the deck is never submerged. Both specimens were fitted with a 100 mm high wooden wall around their perimeter to accumulate the water supplied from above during thawing. The water then spills over the wall, down into the tank.

The freeze-thaw procedure adopted was guided by the ASTM C666-97 standard. All specimens were saturated by submersion in water for at least 24 hours first. The target temperature range of the concrete core during FT cycles was +5 °C to -18 °C (Zero Fahrenheit). Thermocouples were installed in the core of the specimens to monitor their temperature continuously. A typical cycle took 8 hours, including about 6 hours freezing in air and about 2 hours thawing in water (Fig. 2-5(c)). To achieve the desired target core temperatures, the air temperature range, monitored through the environmental chamber controls had to be larger, +13 °C to -25 °C (Fig. 2-5(c)). In fact, the measured core temperature during freezing and thawing varied among all specimens with only ±2 °C but this still permissible. After the specimens completed the FT cycles, they were removed.
from the tank, left to dry, and then tested to failure in a special setup, which is discussed next.

**Structural loading setup and instrumentation:** Figures 2-1(b) and 2-6 show the test setup, which consists of two steel HSS sections tied together using 30 mm steel threaded rods. Vertical threaded rods, 19 mm diameter, were welded to the top flanges of the HSS sections. These rods are intended to fit through the ducts in the beams of the concrete decks to facilitate anchorage of the specimens to this reusable part of the setup. Also, to prevent the decks from sliding laterally, a shallow steel plate was welded to the flange of the HSS section to constrain the concrete beam of the deck. The process starts by fitting the vertical threaded rods through the ducts of the deck and seating the deck on a thin layer of high strength grout placed on the top flanges of the HSS sections. Steel channel sections, oriented parallel to the beams, are then placed above the deck with a thin layer of high strength grout in between, and anchored to the vertical threaded rods using bolts and washers to clamp the deck. The setup design was chosen to provide the proper end constraints against rotation and lateral displacement as recommended by Nelson et al. (2014).

A single load, representing half the axle load, was applied at the centre of the deck using a hydraulic ram and monitored using a 453 kN load cell. The load was applied through a 181x91 mm steel plate (scaled down from the standard 510x255 mm AASHTO patch) resting on a 12.7 mm elastomeric pad. Linear Potentiometers (LPs) were used to measure deflection in both directions at various locations. GFRP panel strains at various locations, in two directions, and the top GFRP rebar strains over the support, were all measured using 5 mm electric resistance strain gauges. Concrete strains on top were measured
using 100 mm PI gauges.

2.3 EXPERIMENTAL RESULTS
Table 2 provides a summary of the test results of all specimens in terms of ultimate loads. The following sections provide details of the test results and the effect of various parameters on the ultimate strength.

2.3.1 Effect of Surface Treatment of FRP Forms
Figure 2-7(a) compares the load-deflection responses of the control specimens S1-S3 with various bond conditions. The figure also shows the three loading and unloading cycles before loading to failure. Specimens S1 and S2 (with adhesive bond and bonded aggregate, respectively, as well as bonded splices) behaved quite similarly and showed higher stiffness after cracking than specimen S3 (without any bond at surface or splice). S3 also showed a 21% lower capacity but higher deformability at ultimate. However, the lower ultimate load of S3 is still 3.9 times higher than the equivalent service load of 24.3 kN at this scale.

Figure 2-8 shows the deflected shape of decks S1-S3 at centerline, at various load levels. The right side of the graph shows the deflection in the transverse direction (i.e. parallel to traffic), while the left side shows the deflection in the longitudinal direction (i.e. deck span direction). Deflected shapes are given at various load levels, namely 35, 70, 105 kN and the peak load of each. The figure shows that deflections converge to a very small value, almost zero, at the edge of the decks in the transverse direction. This confirms the adequate width of 1000 mm selected to ensure a two-way slab action, and to
simulate as close as possible a typical real deck, which is continuous in both directions.

2.3.2 Effect of Freeze-Thaw Cycles

Figure 2-7(b) shows the load-deflection behavior of specimens S4-a, S4-b, S5 and S8 which have been exposed to 300 FT cycles. Specimens S4-a and S4-b with adhesive bond are identical repetitions for confirmation of results, and varied in strength by only 5%, giving reasonable confidence in results. Specimen S5 had bonded aggregate surface treatment while S8 had no surface treatment. By comparing the responses of these specimens to their control counterparts in Fig. 2-7(a), it can be seen that no reduction occurred in ultimate capacity, but to the contrary, a 3-6% increase occurred. This is likely attributed to the increased strength of concrete due to moisture curing during the 100 days in the tank as can be seen in Fig. 2-3 for concrete compressive strengths.

Figure 2-9 shows the load-deflection responses of specimens S6, S7 and S8 after 100, 200 and 300 FT cycles, compared to control specimen S3. After 100 and 200 cycles, a small reduction of about 8% occurred in ultimate load but after 300 cycles, a 3% increase occurred as indicated earlier. Figure 2-10 summarizes the deck strength overtime at various FT cycles. Generally, the observed negative or positive variations in strength are very small and fall within the natural range of statistical variation among identical repetitions (e.g. identical specimens S4-a and S4-b varied by 5%). Therefore, the important observation here is that FT cycles had no negative impact on the FRP SIP forms for concrete decks, even for the decks without any surface treatment or bonded splice.
2.3.3 Effect of Submersion and Form Perforation for Drainage

Figure 2-11 shows the load-deflection responses of specimens S8 (submerged during thawing), S9 (not submerged) and S10 (perforated FRP form and also not submerged) after 300 FT cycles. The figure shows that specimens S9 and S10 achieved the same strength, which was 11% lower than S8 and 8% lower than control S3. It appears then that submersion did not have the hypothesised negative effect of aggravating FT conditions, but rather it might have improved the outcome slightly by helping concrete curing. It is also clear that any weakening effect to the FRP form, from perforation, was not to the extent that it negatively impacts the overall deck strength. This is because at failure of the deck, the stress level in the FRP forms was very small.

2.3.4 Failure Mode

All deck specimens failed by concrete punching shear (Fig. 2-12). Cracks that initiated in the negative moment regions above the girder during the initial cracking cycles propagated in a curved manner as loading progressed. Radial and circumferential cracks occurred around the loading pads. Eventually, punching failure occurred near the peak load and the punching cone was fully contained within the deck. At the underside of the deck, bonded FRP forms differed from unbonded ones at the splice overlap. Figure 2-13 shows the load-vertical slip responses of specimens S1-S3 with various surface treatments and bond conditions at the splice. The slip was measured as the difference in deflection based on two adjacent LPs on both sides of the splice overlap. It can be seen that the slip is quite insignificant in specimens S1 and S2 with adhesive and mechanical bond at splice, whereas in unbonded S3, the slip approached 2 mm at failure.
2.3.5 GFRP and Concrete Strains

Figure 2-14 shows the load-strain responses of the GFRP SIP form in both directions at key locations shown in Fig. 2-14(a). Generally, at ultimate, the GFRP maximum strain occurred in the deck span direction, directly under the load and the nearby region (gauges L1 and L3 in Fig. 2-14(a)). This maximum strain did not exceed 1800 micro-strain, when punching shear failure occurred. From Fig. 2-2(a) it can be seen that this strain is well below the failure strain of the material. For this reason, the small reduction in GFRP strength due to FT cycles (Fig. 2-2(a)) as well as the perforation of the form (Fig. 2-4(d)) did not have any negative impact on the ultimate capacity of the deck. Figure 2-14(c) shows the GFRP strain in the span direction near the support beam (gauge L2). The level of this strain in compression reflects somewhat the degree of deck flexural fixity at supports. For bonded specimens S1, S2, S4 and S5, this strain generally showed the maximum measured compression. On the other hand, the remaining unbonded specimens generally showed a small tension, suggesting that a ‘tie’ action was more dominant than ‘flexural’ action in the GFRP form. It is worth noting that the specimens were sensitive to preclamping but every effort was taken to be consistent. In the direction parallel to traffic, Figs. 2-14(e and f) show that the maximum strains measured in the GFRP form in tension was less than 1000 micro-strain, which is well below the ultimate value given in Fig. 2-2(b). The measured small compression could be attributed to Poisson’s ratio.

Figure 2-15(a) shows the load-concrete compressive strain responses at top surface in a direction parallel to the deck span, near the center. It can be seen that at ultimate, when punching shear failure occurred, the maximum concrete strain ranged from -1000 to -2100 micro-strains, which is less than the concrete crushing strain of
-3000 micro-strains.

Figure 2-15(b) shows the load-GFRP rebar tensile strain responses over the support, at mid width. It can be seen that at ultimate the maximum tensile strain was about 2500 micro-strain, well below the 17000 micro-strain rupture strain of the bar reported by manufacturer.

2.4 SUMMARY
This study examined the effect of aggressive freeze-thaw (FT) cyclic exposure on strength and integrity of bridge decks built using glass fibre reinforced polymer (GFRP) stay-in-place (SIP) structural forms that completely replace the bottom rebar layers. The concern has been whether entrapped moisture may cause ‘frost-jacking’ of the form, negatively impacting deck integrity. Eleven scaled deck specimens were built, each using two flat GFRP plates with T-shape ribs running normal to girders. Both plates spanned the gap between girders and were spliced by overlapping, directly under the load. The study simulated various surface treatments of the form, namely no treatment at all, adhesive bond to freshly cast concrete, and coarse aggregates bonded to the forms. Also, unbonded and bonded lap splices of the forms were tested. The decks were subjected to three cracking load cycles before being saturated and subjected to up to 300 FT cycles at +5 °C to -18 °C core temperatures. Freezing was in air while thawing was by water. Some specimens were thawed without being submerged and one specimen had perforated forms for drainage. Specimens of various splice and surface treatments survived the 300 FT cycles. Subsequent testing showed no reduction in ultimate load or stiffness, relative to control specimens, despite the 23% reduction in tensile strength and
11% in modulus of GFRP coupons exposed to the 300 FT cycles. This is because failure of the decks was governed by concrete punching shear. Decks with untreated forms and unbonded splices showed 21% lower capacity than treated and bonded ones, even without any FT exposure, but this lower capacity was still 3.9 times higher than the equivalent design truck service load at this scale.

Table 2-1: Summary of test matrix

<table>
<thead>
<tr>
<th>Seam</th>
<th>Panel Surface Treatment</th>
<th>Freeze-Thaw Cycles</th>
<th>Concrete Strength (MPa)</th>
<th>Ultimate Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>x</td>
<td>x</td>
<td>24.8</td>
<td>121.3</td>
</tr>
<tr>
<td>S2</td>
<td>x</td>
<td>x</td>
<td>24.8</td>
<td>118.0</td>
</tr>
<tr>
<td>S3</td>
<td>x</td>
<td>x</td>
<td>24.8</td>
<td>94.6</td>
</tr>
<tr>
<td>S4-a</td>
<td>x</td>
<td>x</td>
<td>300</td>
<td>27.3</td>
</tr>
<tr>
<td>S4-b</td>
<td>x</td>
<td>x</td>
<td>300</td>
<td>27.3</td>
</tr>
<tr>
<td>S5</td>
<td>x</td>
<td>x</td>
<td>300</td>
<td>27.3</td>
</tr>
<tr>
<td>S6</td>
<td>x</td>
<td>x</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>S7</td>
<td>x</td>
<td>x</td>
<td>200</td>
<td>23.4</td>
</tr>
<tr>
<td>S8</td>
<td>x</td>
<td>x</td>
<td>300</td>
<td>27.3</td>
</tr>
<tr>
<td>S9*</td>
<td>x</td>
<td>x</td>
<td>300</td>
<td>27.3</td>
</tr>
<tr>
<td>S10*</td>
<td>x</td>
<td>x</td>
<td>300</td>
<td>27.3</td>
</tr>
</tbody>
</table>

* Thawing by water but without submersion
Figure 2-1: Schematic diagram of test specimens

Figure 2-2: GFRP SIP Stress-Strain Curves
Figure 2-3: Variation of concrete strength with time

Figure 2-4: Surface preparations of FRP SIP form
Figure 2-5: Environmental exposure arrangements and protocol

Figure 2-6: Test Setup
Figure 2-7: Load – deflection responses of specimens with and without FT exposure

Figure 2-8: Deflected shapes of specimens S1, S2 and S3 in both directions
Figure 2-9: Load – deflection responses of specimens without surface treatment after various Freeze-Thaw cycles

Figure 2-10: Variation of deck strength with Freeze-Thaw cycles with different configurations
Figure 2-11: Load-deflection responses of specimens with and without drainage holes compared to submerged specimen after 300 FT cycles

Figure 2-12: Punching shear failure, top view
Figure 2-13: Slip at GFRP form splice in decks with various surface preparations
Figure 2-14: Load-strain responses in FRP SIP forms at various locations
a) Concrete compressive strain

b) Top GFRP rebar strains at support

Figure 2-15: Load- top strains responses at different locations
Chapter 3: The Effect of the Shape of FRP Stay-in-Place Structural Form on Durability of Concrete Members under Freeze-Thaw Cycles

3.1 INTRODUCTION
Over the past decades, deterioration of infrastructure has been widely recognized by scholars throughout cold climate regions, particularly North America, as a major and costly problem. The problem is quite complex in that it involves various damaging mechanisms arising from different causes, namely de-icing salts causing corrosion in bridges, freeze-thaw (FT) cycles, and wet-dry cycles. Fibre reinforced polymer (FRP) reinforcements have been introduced in various shapes and forms as a potential solution to this problem, due to their inability to corrode. This includes internal reinforcing bars, and sheets and plates for external bonding to retrofit structures. Also, structural shapes have emerged as stay-in-place (SIP) structural forms for rapid construction of concrete structures, taking various shapes such as ribbed plates (Hall and Mottram, 1998). A key consideration in the SIP structural form system is the interface condition between the concrete and form, which affects the bond. Durability of this bond under mechanical loading or environmental cycles becomes a concern. In particular, under FT cycles, ‘frost-jacking’ might cause separation of the SIP form.

The concept of FRP SIP forms has been studied for beam and slab applications. A study by Honickman et al. (2009) explored various bond mechanisms at the interface between glass-FRP (GFRP) flat plates and concrete slabs, namely wet adhesive bond to freshly

---

2 This chapter has been submitted for publication as the following journal paper:
cast concrete, a layer of course aggregates bonded to the plate prior to casting, and mechanical shear connectors using threaded GFRP or steel rods. The study found that adhesive bond and bonded aggregates showed superior strength and stiffness and better distribution of cracks. Although no slip occurred in this type of bond, sudden shear-bond failure governed at ultimate. Honickman and Fam (2009) extended this concept to a hat-shape GFRP form for girder applications, where voided concrete cores were explored for box sections.

One of the most promising applications of the concept of SIP structural forms is in bridge decks due to the rapid construction feature of the system, which minimizes road closure and traffic delays. Several studies and field applications have been accomplished in this area, under static load (e.g. Berg et al. 2005, Nelson and Fam, 2013). While durability of externally bonded FRP laminates for concrete retrofitting applications has been studied sufficiently, including exposure to FT cycles (e.g. Karbhari and Engineer, 1996, and Green et al. 2000), very limited work has been done on durability of SIP form systems. A recent study on bridge decks with GFRP SIP forms exposed to aggressive FT cycles showed almost no reduction in ultimate strength (Boles et al. 2014). It is possible that this was attributed to the end constraints of the deck being cast on precast concrete girders with protruding stirrups embedded into the ends of the deck as in typical construction. But it is also possible that the specific SIP form geometry has a significant effect on the response of the deck to FT exposure.

Previous studies have addressed FRP SIP forms of different configurations but did not directly compare them, particularly in terms of durability under aggressive freeze-thaw exposure. One of the concerns is that entrapped moisture at the interface between
the FRP SIP form and concrete might cause ‘frost-jacking’ which may compromise the bond integrity. In this study, two different configurations of the FRP forms are compared, namely a flat plate with T-shape ribs and a corrugated plate (Fig. 3-1(a)). It is hypothesized that the ribbed flat plate would be superior to the corrugated plate, as any frost-jacking effect would be counteracted by the gripping resistance of the T-shape ribs anchored into the concrete.

3.2 EXPERIMENTAL PROGRAM
The following sections provide details of the experimental program, including test specimens and parameters, materials, fabrication, cold room setup, structural test setup and instrumentation.

3.2.1 Test Specimens and Parameters
The experimental program included nine one-way concrete slab specimens, 2 m long each, referred to as S1 to S9 (Table 3-1). The slabs were built using two different types of GFRP stay-in-place structural forms, namely; a flat plate with T-shape ribs, and a corrugated plate. Figure 3-1(a) shows the two cross-section configurations. These two types of forms were used in bridge deck applications and tested under static and fatigue loading (Nelson and Fam, 2013, Fam and Nelson, 2012, and Richardson et al. 2014). In this study, the durability of the two systems is compared using the simply supported one-way slab specimens under four-point bending (Fig. 3-1(b)). This setup provides critical conditions that promotes bond and/or shear failure, as the shear span-to-effective depth ratios are 3.3-3.8 (Table 3-1). The objective is to examine the effects of form shape and geometry along with surface preparation technique on the strength of bond-critical specimens with and without exposure to FT cycles.
Table 3-1 provides a summary of test matrix. Specimens S1-S5 were control specimens tested to failure at room temperature, including the two different form types, namely corrugated plate (specimens S1-S3) and ribbed plate (S4-S5). Specimens S1-S3 had a 270x225 mm cross-section, with GFRP reinforcement ratio of 3.5%, while S4-S5 had a cross-section of 290x237 mm, with GFRP reinforcement ratio of 2.34%, where the reinforcement ratio $\rho_f$ is defined as the ratio of the cross sectional area of GFRP panel only-to-concrete cross-sectional areas ($A_f/bd$). The two beam configurations were designed to have a similar stiffness of their reinforcement ratio $(E_f \rho_f)$ of 0.67 GPa and 0.69 GPa, for the corrugated and ribbed panels, respectively, where $E_f$ is the GFRP longitudinal modulus. It is well established that the quantity $(E_f \rho_f)$ of longitudinal reinforcement affects the shear strength of concrete beams reinforced by FRP as it directly influence the diagonal crack width, which interns affects aggregate interlock along the inclined plane of the crack.

Different surface preparation methods were examined in both configurations, namely, ‘nothing’ other than surface cleaning of the GFRP form prior to concrete casting (specimens S1 and S4), applying an adhesive layer of epoxy shortly before casting, which bonds to concrete (S2) and applying a layer of adhesively bonded coarse aggregates to the GFRP form (S3 and S5). Specimens S6-S9 were exposed to 300 FT cycles where the core temperature range during FT cycles was +5 °C to -18 °C. Specimens S6 and S7 were identical to S2 and S3, respectively, while S8 and S9 were identical to S4 and S5, respectively.
3.2.2 Materials

**GFRP ribbed flat panels and corrugated panels:** Commercially available pultruded GFRP panels made from E-glass fibres and polyester resin were used. The first type was 4.2 mm thick flat plate with T-shaped ribs of the same thickness, spaced at 100 mm. The second type was 4.2 mm thick corrugated panels with pin and eye end connections for splicing purposes. Five standard coupons (250x25 mm) were cut from each panel, and tested in tension according to ASTM D3039/D3039M-08. Figure 3-2 shows the stress-strain curves of the two panel materials. The average tensile strength and elastic modulus of the ribbed panel material was 352 MPa (with a standard deviation of 16.5 MPa) and 29.5 GPa, respectively, while the tensile strength and modulus of the corrugated panel material was 327 MPa (with 15 MPa standard deviation) and 18.9 GPa, respectively.

**GFRP rebar:** V-Rod #2 GFRP rebar mats of 6.35 mm diameter were installed in all specimens as top reinforcement for shrinkage and thermal crack control. The tensile strength and elastic modulus provided by the manufacturer were 784 MPa and 46.1 GPa, respectively.

**Concrete:** The concrete mix used in all slabs had 10 mm coarse aggregates and was designed to have a 7% air entrainment, a 200 mm slump and compressive strength of 35 MPa. At the time of testing the control specimens, 15 concrete cylinders were tested in compression according to ASTM C39 (2010) and showed a strength range of 36.4-46.6 MPa with an average of 41.8 MPa and a standard deviation of 3.4 MPa. Additional 15 cylinders were exposed to the 300 FT cycles and were tested after. They showed a similar strength range of 38.2-45.9 MPa with an average of 41.4 MPa and a standard deviation of 2.05 MPa.
Coarse aggregates: 4-9 mm pure silica stones were used in the ‘bonded aggregate’ system and were bonded to the GFRP panel using epoxy adhesive.

Epoxy adhesives: Two types of adhesives were used. When bonding the silica stones to the surface of the FRP panel, Sikadur 30 was used. It is a high modulus, high strength, and high viscosity epoxy paste adhesive with a 20.6 MPa bond strength as reported by the manufacturer. For bonding freshly cast concrete to the FRP panels, Sikadur 32 Hi-Mod, a high modulus, and low viscosity epoxy adhesive with manufacturer reported bond strength of 13.1 MPa was used to coat the surface with a thin layer, shortly before concrete casting.

3.2.3 Fabrication of Test Specimens
The GFRP panels were cut to the design length of specimens, 2 m each. The panel surface was cleaned with acetone prior to applying the specific surface preparation and bond technique. For the specimens without surface preparation (Fig. 3-3(a)), concrete was cast directly onto the forms. For the bonded aggregates surface treatment, a layer of the viscous epoxy resin was applied to the horizontal surface only (i.e. the bottom flange) of the GFRP panels, followed by a layer of the coarse aggregates pressed into the adhesive layer (Fig. 3-3(b)). From a practical standpoint, it would be difficult to bond the aggregates to vertical or inclined surfaces. Once the epoxy has cured, concrete was cast onto the prepared SIP forms. For the wet adhesive bond system, the thin epoxy coating was applied to the entire surface of the GFRP form using a conventional brush, like paint (Fig. 3-3(c)). The concrete was then cast onto the forms within 15 minutes to bond to the wet adhesive. In all specimens, prior to concrete casting, additional formwork was
necessary to build up the cross-section shape of the specimens. Thermocouple gauges were also secured in place at the center of the core of the specimens before casting and the top reinforcement mesh was also secured in place. After concrete hardening, the specimens were moist-cured for seven days.

3.2.4 Environmental Chamber Setup and Instrumentation

A special wooden tank was built to accommodate the specimens during FT cycles and was placed inside an environmental chamber (Fig. 3-4(a)). The tank was lined with a rubber sheet to prevent water leakage and was fitted with a drain at the lowest point and an overflow outlet near the top. Specimens were stacked in the tank such that during the thawing period, when the tank is filled with water, they are completely submerged.

The freeze-thaw procedure used was guided by the ASTM C666-97 standard. All specimens were saturated by submersion in water for at least 24 hours first, before applying the 300 FT cycles. The target range of temperatures of the concrete core during FT cycles was +5 °C to -18 °C. A typical cycle took 9 hours and 20 minutes, including about 7 hours freezing in air and about 2 hours thawing in water (Fig. 3-4(b)). In order to achieve the desired core temperatures, the air temperature range which is monitored through the environmental chamber controls had to be larger, +13 °C to -25 °C (Fig. 3-4(b)). After the specimens completed the FT cycles, they were removed from the tank, left to dry, and then tested to failure in a special setup, discussed next.

3.2.5 Structural Testing Setup and Instrumentation

Figure 3-1(b) shows a schematic of the four-point bending test setup. Pictures of the
setup are also shown in Fig. 3-5(a) for the specimens with ribbed flat plate and in Fig. 3-5(b) for the specimens with corrugated plate. The simply supported span and constant moment zones were 1800 mm and 300 mm, respectively. The specimens were tested to failure under stroke control using a 900 kN Reihle testing machine at a rate of 1 mm/min. One Linear Potentiometer (LP) was used at mid-span to measure the deflection. Also, two LPs were installed horizontally at each end of the specimen to measure any relative slip between the GFRP plate and concrete. Longitudinal strains were measured at mid-span, on both the bottom surface of the GFRP form, and the top surface of concrete, using 5 mm and 65 mm electric resistance strain gauges, respectively. Additional one PI gauge of 100 mm displacement transducers was used at the center of concrete top surface to verify concrete strain.

3.3 EXPERIMENTAL RESULTS

Table 3-2 provides a summary of the test results of all specimens, in terms of peak loads, maximum strains in GFRP and concrete, mid-span deflections at peak loads, load at 0.1 mm slip, and slip values at peak loads. Figure 3-6 shows the load-mid-span deflection responses. Figures 3-7 and 3-8 show the load-strain responses of GFRP form in tension and concrete in compression, respectively. Figure 3-9 shows the load-slip responses between the GFRP form and concrete. Figure 3-10 shows the failure modes and Fig. 3-11 compares the two different GFRP form configurations in terms of their normalized responses in shear. The following sections discuss in detail the test results.
3.3.1 Effect of Surface Treatment of GFRP Forms in Control Specimens

**Corrugated forms:** Specimen S1 had no surface treatment other than cleaning, while S2 and S3 had wet adhesive and bonded coarse aggregates, respectively, prior to casting. All specimens were kept at room temperature as control specimens. Figure 3-6(a) shows that the cracking loads of bonded specimens S2 and S3 are quite similar and are significantly higher than that of unbonded specimen S1, due to the contribution of the GFRP section to the transformed uncracked moment of inertia of the section. Upon first cracking, specimen S1 loses strength completely, while S2 and S3 show significantly higher loads, 4.8 and 3.8 times, respectively, at a reduced stiffness. The early failure of S1 is not surprising because of the complete lack of bond and the shape of the GFRP form, which provides no interlocking with concrete. For this reason, there was no point of testing the configuration of S1 under FT cycles. Despite their similar post-cracking stiffness, the ultimate capacity of S2 was 26% higher than S3. This is attributed to the difference in bond capacity at their respective concrete-form interfaces. The wet adhesive in S2 was applied to both the horizontal and inclined surfaces because it was practically feasible, whereas in S3 bonded aggregates could only be applied to the horizontal surface.

Figure 3-7(a) shows that the maximum measured tensile strain in the corrugated GFRP form was about 4500 micro-strain, which is significantly lower than the ultimate strain of the material (Fig. 3-2). It is also noted that upon first cracking, the GFRP strain in S3 is considerably higher than S2 at any given load. This is attributed to the lack of bond at the inclined surface of the form with bonded aggregates, which provides less control of crack width and distribution, compared to S2 with full surface bond. The better crack control leads to finer and better distribution of cracks. It is also noted that under bending and
curvature effect of this hat-shape GFRP section, the inclined sides tend to buckle slightly outwards, separating from concrete, due to lateral torsional buckling, which reduces their effectiveness. Figure 3-8(a) shows that the maximum compressive strain in concrete was about -1100 micro-strain, which is well below the -3000 crushing strain of concrete. Figure 3-9(a) shows a slip of about 0.28 mm occurred between the form and concrete throughout the loading history, suggesting excellent bond in specimens S2 and S3. It should be noted that this slip was insignificant compared to measured slip of the identical beam after exposure to F/T cycles.

**Ribbed forms:** Specimens S4 had no surface treatment while S5 had bonded aggregates and both were control specimens. Similar to beams S1-S3, Fig. 3-6(b) shows a significantly lower cracking load for S4 compared to S5. However, unlike S1, the load after cracking of S4 dropped only slightly due to loss of stiffness but started rising again to a load level significantly higher than the cracking load. Once the peak load was reached, it remained fairly stable over a large deflection range. Figure 3-7(b) clearly shows that under the relatively constant load, the tensile strain in the GFRP form of S4 did not increase much as deflection increased. This is attributed to the onset of slip at peak load (Fig. 3-9(b)) and its rapid increase. Two significant aspects may be noticed here: first, despite the lack of any surface preparation, specimen S4 was able to reach a load considerably higher than cracking load before slip occurred, and secondly, despite the excessive slip the peak load was maintained over large deflection, producing a ‘pseudo-ductile’ response. This is clearly attributed to the T-shape ribs providing some mechanical interlock to concrete, which is enhanced by curvature.
Specimen S5, on the other hand, showed a considerably higher, 2.3 times, ultimate load than S4 but did not show the apparent pseudo-ductility that S4 showed, due to the enhanced bond. Figure 3-7(b) shows that the maximum GFRP tensile strain reached was about 3500 micro-strain only, significantly lower than the ultimate value (Fig. 3-2). Figure 3-8(b) also shows that the maximum concrete compressive strain was about -1000 micro-strain only.

**Failure modes:** Specimen S1 with no bond failed immediately upon first cracking as debonding occurred (Fig. 3-10(a)). The specimen broke completely into two pieces. Specimens S2 and S3 with wet adhesive and bonded aggregates exhibited flexural and flexural-shear cracks. Eventually they both failed in concrete shear through diagonal tension cracking (Fig. 3-10(b)). A secondary debonding failure of the sides of the GFRP form also occurred. Specimen S4 with no surface treatment exhibited one flexural crack and eventually failed by horizontal shearing of one of the GFRP T-shape ribs from the flat GFRP base (Fig. 3-10(c)). Specimen S5 with bonded aggregates exhibited several flexural and flexural-shear cracks. Eventually it failed in shear by diagonal tension cracking of concrete (Fig. 3-10(d)). No bond failure was observed in this specimen.

### 3.3.2 Effect of Freeze-Thaw (FT) Cycles

**Corrugated forms:** Specimens S6 and S7 with wet adhesive and bonded aggregates interfaces, respectively, were exposed to 300 FT cycles then tested to failure. Figure 3-6(a) shows that FT exposure has resulted in 21% and 18.4% reduction in the strength of specimens S6 and S7, respectively, relative to their control counterparts S2 and S3. The reductions appear to be similar despite the different surface treatment. On the other hand,
the stiffness as represented by the slope of load-deflection curves appear unchanged after FT exposure. Figure 3-7(a) clearly shows that the GFRP strains of S6 are larger than S2 at any given load. In fact, the cracking load of S6 is smaller than S2. The figure also shows that S7 appears to have been already cracked before the test. It is possible that this might have occurred during lifting and handling of the beam into and out of the aging tank in the environmental chamber. Figure 3-9(a) shows that the relative slip at peak load was about 0.4 mm for both S6 and S7, larger than their control counterparts.

**Ribbed forms:** Specimens S8 and S9 with no surface treatment and bonded aggregates, respectively, were exposed to 300 FT cycles then tested to failure after. Figure 3-6(b) shows that the FT exposure had absolutely no effect on their ultimate strengths and stiffness. Their ultimate loads and the entire load-deflection curves match almost exactly those of their control counterparts S4 and S5. Similarly, Figs. 3-7(b), 3-8(b) and 3-9(b) confirm this observation of almost identical behaviour of both exposed and control specimens.

**Failure modes:** Specimens S6 and S7 with wet adhesive and bonded aggregates, which were exposed to FT cycles, had bond failure between the corrugated form and concrete (Fig. 3-10(e)). It is worth noting that their control counterparts S2 and S3 had a primary concrete shear failure and a secondary bond failure (Fig. 3-10(b)). This indicates that FT exposure has caused significant damage to bond, leading to a change in failure mode in corrugated form system. Similar to specimen S4, specimen S8 with no surface treatment, and exposed to FT cycles, failed by horizontal shearing off failure of the T-shape rip from the base (Fig. 3-10(c)). Also, specimen S9 with bonded aggregates failed similar to S5, by concrete diagonal tension shear (Fig. 3-10(d)). Unlike corrugated forms, FT exposure
did not cause any change in failure modes of ribbed forms.

3.4 SUMMARY
This study investigated the effect of aggressive regime of 300 freeze-thaw (FT) cycles, at a core temperature range of +5 °C to -18 °C on the structural behaviour and bond integrity of one-way concrete slabs cast onto glass fibre reinforced polymer (GFRP) stay-in-place (SIP) structural forms. The study aims at comparing two configurations of the SIP forms, namely a flat plate with T-shape ribs and a corrugated plate, under the potential ‘frost-jacking’ effect arising from FT cycles. The study explored specimens with no surface treatment, wet adhesive bonding to freshly cast concrete, and bonded coarse aggregates to enhance roughness of SIP form. It was clearly shown that flat-ribbed form specimens are superior to the corrugated form ones, as no loss in strength occurred after the FT exposure, whereas the corrugated form specimens lost 18-21%. This is attributed to the anchorage advantage provided by the T-shape rib embedment into concrete. Specimens with untreated corrugated forms showed strengths that are only 21-26% of treated ones. For the flat-ribbed form specimens, the one with the untreated form had 44% the strength of the one with bonded aggregates
Table 3-1: Summary of test matrix

<table>
<thead>
<tr>
<th>Panel types</th>
<th>Bond Mechanism</th>
<th>Cross sectional details</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bonded Aggregate Width b (mm)</td>
</tr>
<tr>
<td>Corrugated</td>
<td>No bonding</td>
<td>270.0</td>
</tr>
<tr>
<td>Ribbed</td>
<td>Adhesive bond</td>
<td>290.0</td>
</tr>
</tbody>
</table>

Control Specimens
- S1: Corrugated x Ribbed x
- S2: x x
- S3: x x
- S4: x x
- S5: x x

Specimens Subjected to 300 F/T cycles
- S6: x x
- S7: x x
- S8: x x
- S9: x x

* shear span (a)=0.75 m

Table 3-2: Summary of test results

<table>
<thead>
<tr>
<th>Peak load (KN)</th>
<th>Maximum strains</th>
<th>Mid span deflection at peak load (mm)</th>
<th>Load at 0.1 mm slip (KN)</th>
<th>Slip at peak load (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top (micro)</td>
<td>Bottom (micro)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>17.9</td>
<td>-107</td>
<td>0.47</td>
<td>-</td>
<td>0.01 Bond failure</td>
</tr>
<tr>
<td>S2</td>
<td>85.8</td>
<td>-1060</td>
<td>10.49</td>
<td>50.4</td>
<td>0.28 Concrete shear (diagonal) - Bond failure</td>
</tr>
<tr>
<td>S3</td>
<td>68.1</td>
<td>-1013</td>
<td>8.08</td>
<td>54.6</td>
<td>0.18 Concrete shear (diagonal) - Bond failure</td>
</tr>
<tr>
<td>S4</td>
<td>48.4</td>
<td>-219</td>
<td>4.96</td>
<td>29.8</td>
<td>0.32 Horizontal shear of GFRP rib from base</td>
</tr>
<tr>
<td>S5</td>
<td>111.2</td>
<td>-978</td>
<td>8.04</td>
<td>29.2</td>
<td>3.63 Concrete shear (diagonal)</td>
</tr>
<tr>
<td>S6</td>
<td>67.8</td>
<td>-890</td>
<td>7.50</td>
<td>35.4</td>
<td>0.38 Bond failure</td>
</tr>
<tr>
<td>S7</td>
<td>55.6</td>
<td>-700</td>
<td>7.15</td>
<td>34.8</td>
<td>0.41 Bond failure</td>
</tr>
<tr>
<td>S8</td>
<td>48.1</td>
<td>-202</td>
<td>5.76</td>
<td>33.5</td>
<td>0.50 Horizontal shearing of GFRP rib from base</td>
</tr>
<tr>
<td>S9</td>
<td>112.3</td>
<td>-1086</td>
<td>7.81</td>
<td>43.0</td>
<td>0.53 Concrete shear (diagonal)</td>
</tr>
</tbody>
</table>
Figure 3-1: Schematic diagram of test specimens, setup and instrumentation
Figure 3-2: GFRP SIP form stress-strain curves

Figure 3-3: Various surface preparations
a) Specimens in aging tank

Figure 3-4: Conditioning setup showing the water tank inside cold room

b) Freeze-thaw cycles protocol

Figure 3-5: Test setup
a) Corrugated panel specimens  

b) Ribbed panel specimens

**Figure 3-6: Load – deflection responses**

a) Corrugated panel specimens  

b) Ribbed panel specimens

**Figure 3-7: Load – FRP bottom strain responses**
Figure 3-8: Load – concrete top surface strain responses

Figure 3-9: Load – slip responses
Figure 3-10: Failure modes
4.1 INTRODUCTION

Fibre reinforced polymers (FRPs) have seen widespread application in bridges with the aim of better providing durability than conventional steel reinforcement. These FRP materials are quite versatile. Their applications have included external bonding of sheets and laminates for rehabilitation, reinforcing bars for internal reinforcement of concrete decks and structural shapes for girders and decks (Bakis el al. 2002). Bridge decks in particular suffer the most from deterioration due to the direct application of de-icing salts, leading to steel corrosion. While FRP reinforcements are corrosion-resistant, they could be vulnerable to other deterioration mechanisms related to moisture and temperature. Absorbed moisture can lead to permanent physical and chemical changes in the molecular structure, within the fibres and matrix themselves, or at the interface. As the unit volume increases due to water ingress, the adhesion between the fibres and matrix is weakened as micro-cracks develop at the interface, leading to bond degradation and overall loss of mechanical properties (Schutte, 1994). Moreover, the absorbed moisture reduces the glass transition temperature ($T_g$) of the polymers, which represents the temperature that the phase of the material changes from glassy to rubbery (Kumar and Gupta, 1998).

---

4 This chapter has been submitted for publication as the following journal paper: Fam, A., Boles, R. and Robert, M. (2014) “Durability Study on Pultruded FRP Sections used in Bridge Deck Application”, ASCE Journal of Bridge Engineering, Under review
5 Robert, M. contributed by performing microstructure tests namely DSC, SEM and FTIR
Chen et al. (2007) conducted durability tests on glass-FRP (GFRP) bars immersed in various solutions namely: water, alkaline solution and saline solution at a temperature range of 20-60°C. It was determined that, after 240 days, the rate of deterioration reflected by the reduction in tensile strength of the bars increases with temperature. Elevated temperatures were reported to induce more stress in the fibres, contributing to the decrease in GFRP bar strength (Vijay and GangaRao, 1999). Another series of durability tests on GFRP laminates with polypropylene thermoplastic resin were conducted after immersion in both tap water and 3% (by weight) salt-solution (Robert et al. 2010). This concentration simulates closely the conditions of chlorides in de-icing salts on roads and bridges in the winter. The results after 168 days showed a more significant degradation of flexural properties in the salt-solution than in tap water.

Extensive research was also conducted on durability of GFRP laminates used for rehabilitation of structures by means of adhesive bonding, including both conventional epoxies (Kajorncheappunngam et al. 2002) and new bio-resins (Eldridge and Fam, 2013). Cromwell et al. (2011) examined the durability of GFRP and CFRP fabrics and CFRP plates immersed in water, saltwater, alkaline solution, dry heat, diesel fuel, weathering, and freeze–heat exposure. After conditioning, the samples were tested in tension, shear, and bond. The results showed that, the glass transition temperature ($T_g$) of this particular resin may be approached at 60°C and as such the mechanical properties would be markedly changed. Also, the study indicated that the most appropriate method to assess the extreme environmental effects on FRPs is the exposure to saltwater and alkaline solutions. Generally, it is well established through previous research that
increasing the temperature is an effective way to accelerate the aging process of composites and that a 3% NaCl salt concentration represents closely conditions on bridges with regard to the use of de-icing salts. Also, the Arrhenius model is generally used to predict the property retention of composites over service life, using results from accelerated aging (Bank et al. 2003).

Most durability studies have focused on internal FRP reinforcing bars and externally bonded FRP sheets and laminates for rehabilitation. This study is focused on durability of FRP pultruded structural sections used in bridge deck applications. The pultruded sections may be used as stay-in-place structural forms for concrete (Figs. 4-1(a and b)), or as an all-FRP deck (Fig. 4-1(c)). The two sections studied are flat GFRP plate with T-shape ribs and a corrugated GFRP plate. Standard coupons were aged at 23°C, 40°C and 55°C temperatures in three separate solution baths including 3% salt, for up to 224 days and then tested in tension. The tensile strength retentions and Young’s moduli were assessed periodically. Rigorous statistical analysis of the results was performed. Additionally, micro-structure analyses, including Differential Scanning Calorimetry (DSC), Scanning Electron Microscopy (SEM) and Fourier Transform Infrared Spectroscopy (FTIR) tests were conducted to provide additional assessment of degradation, including the effect on glass transition temperatures.

4.2 EXPERIMENTAL PROGRAM
The following sections provide details of the experimental program, namely a description of the materials used, test specimens and parameters, fabrication process, aging setup and
procedure, and the tension test setup and instrumentation.

4.2.1 Materials

**GFRP test coupons:** Test coupons were cut from two different types of GFRP pultruded sections. The first one is a 4.2 mm thick flat plate with 4.2 mm thick T-shape ribs, similar to that shown in Fig. 4-1(a), and is referred to in this study as R-GFRP. The section is pultruded using E-glass fibres with polyester fire retardant resin and is commercially available for flooring and decking applications. This section configuration has been studied as stay-in-place structural forms for concrete bridge decks supported by precast concrete girders and showed exceptional performance (Nelson and Fam, 2013). The second section is a 4.2 mm thick corrugated GFRP plate (Fig. 4-1(b)), and is referred to in this study as C-GFRP. It is also pultruded using E-glass fibres with regular polyester resin. While this section is originally produced as a sheet pile with pin-and-eye connections, it showed excellent performance as stay-in-place structural form for concrete deck, from both a structural performance and an ease of fabrication stand points, particularly with regard to the convenient pin-and-eye splice (Fam and Nelson, 2012). Concrete decks using both GFRP configurations also showed excellent fatigue performance and survived at least three million loading cycles under service loads with maximum dynamic allowance (Richardson et al. 2013).

**End tabs for gripping:** End-tabs were used to prevent grip failure of the coupons. The tabs were fabricated using wet layup of unidirectional E-glass fabrics wetted with low viscosity epoxy resin (Tyfo® S Epoxy-resin), and were left to cure between two flat rigid
boards. The tabs were then cut to size and were adhesively bonded to the coupon ends using a viscous (paste-like) epoxy (Sikadur 30).

4.2.2 Test Specimens and Parameters
Based on ASTM D3039 (2008), a total of 170 standard coupons (250x25x4.2 mm) were prepared and tested in this study, 85 from R-GFRP and 85 from C-GFRP (Fig. 4-2). For each parameter studied, five coupons were tested for repetition, to establish a reasonable statistical average. Table 4-1 provides a summary of the test matrix. For each GFRP type, five control coupons were tested at room temperature without any exposure. Seventy five coupons were split into three groups of 25 coupons each, and were aged at 23°C, 40°C and 55°C temperatures in three separate salt solution baths. As indicated earlier, elevating temperature is a well-established procedure to accelerate the aging process of composites (Bank et al., 2003). At the same time, the elevated temperatures were selected to be below the glass transition temperature ($T_g$) of the resin, to avoid post-curing of the coupons, which affects mechanical properties, making the comparison to control specimens not possible (test results of glass transition temperatures are reported later)

The salt (NaCl) concentration was 3% by weight, to simulate deicing conditions on bridges. Within each group of specimens, five coupons were removed from the bath at various time intervals, namely 14, 33, 97, 150 and 224 days, and tested in tension to failure to establish property retention. Finally, another five control coupons from each GFRP type were post-cured in a special oven at 90°C for 48 hours and then tested to failure in tension. These additional ancillary tests were intended to examine the effect of post-curing on tensile strength of the materials.
4.2.3 Fabrication of Coupons

The coupon specimens were fabricated and tested according to ASTM D3039 (2008) standards. They were cut from large panels, similar to those shown in Figs. 4-1(a and b), as 250x25 mm rectangular strips in the direction of the primary longitudinal fibres, using a diamond tip blade. The coupons were then aged in the salt solutions. At the end of their exposure periods, the coupons were removed from the tanks and dried in air. The prefabricated 25x56 mm GFRP end tabs, made using wet layup, were then adhered to the coupon ends for gripping purposes, using the viscous epoxy adhesive and left to cure (Fig. 4-2). The C-GFRP coupons had a very smooth surface. To enhance their bond to the tabs, the surface was lightly roughened using sand paper before applying the adhesive.

4.2.4 Test Setups, Procedures and Instrumentation

Aging Setup: The aging process followed the guidelines described by (Bank et al. 2009; Eldridge and Fam, 2014) to determine the long-term durability of FRP composites. The coupons were aged by immersion in the salt solutions in three separate tanks at three different temperatures. The two tanks used for the 40°C and 55°C tests, respectively, were heated by a screw-plug heaters on the sides of the tanks and controlled by electronic temperature controllers (Fig. 4-3(a)). These two tanks were also insulated around the sides and the top cover to reduce the amount of heat and moisture that would escape. Also, each tank was elevated above the ground by placing them on wooden pallets to keep the heat from escaping. The two heated tanks had circulation pumps to circulate the hot water in them. The third tank contained water at room temperature and was sealed with a plastic sheet on top. No insulation was required because this tank was kept in a controlled room with a constant 23°C air temperature.
**Tension Test Setup:** Control and aged coupons were tested to failure in tension using an Instron universal testing machine (model 8808) with hydraulic grips (Fig. 4-3(b)). The grip pressure was set to 2.76 MPa (400 psi) for the R-GFRP coupons, and 4.14 MPa (600 psi) for the C-GFRP coupons. Based on several initial trials, these gripping pressures were established as the minimum required to prevent slip and reach tension failure of the coupon far from the tabs. A constant stroke rate of loading of 2 mm/min was used as per ASTM D3039 (2008). The strain was measured using a calibrated extensometer for all coupons, but was removed at approximately 55% of the predicted failure load to prevent its damage.

**Micro-Structure Assessment**
Additional techniques were also used to characterize the potential degradation of the glass fibres, the polymer matrix or the interfaces, if any. The characterized samples were cut from control and aged R- and C-GFRP coupons. The aged coupons were those exposed to the highest temperature of 55°C, for almost the longest durations of exposure; 214 and 204 days for the C-GFRP and R-GFRP, respectively. Differential Scanning Calorimetry (DSC) was used to obtain information on the thermal behaviour of the polymer matrix. In particular, the glass transition temperature \( T_g \) of the composites was determined in accordance with the ASTM E1356 (2008) standard. A decrease in \( T_g \) of conditioned samples would generally be an indication of a plasticizing effect or a chemical degradation.

Scanning Electron Microscopy (SEM) observations and image analysis were also performed on a JEOL JSM-840A SEM to observe the microstructure of specimens before
and after aging in salt solution. All specimens observed in the SEM were first cut, polished and coated with a thin layer of gold–palladium by a vapour-deposit process.

Finally, Fourier Transform Infrared Spectroscopy (FTIR) was also performed to characterize any irreversible chemical degradation such as hydrolysis of the polymer matrix. FTIR spectra were recorded using a Nicolet Magna-550 spectrometer equipped with an attenuated total reflectance (ATR) device. Fifty scans were acquired with an optical retardation of 2.5 mm to yield a resolution of 0.4 mm⁻¹.

4.3 EXPERIMENTAL RESULTS AND DISCUSSION

Table 4-2 provides a summary of all test results, in terms of the mean tensile strength and standard deviation, and the mean Young’s modulus and standard deviation. The table also provides the percentage reduction in tensile strength relative to control specimens. In Table 4-2, specimens are given identification that describes the specific parameters they represent. For example: R-40-150 represents the five R-GFRP (ribbed panel) coupons aged at 40°C for 150 days. The following sections give specific details regarding the long-term effect of the environmental aging on the mechanical and micro-structure properties of the R- and C-GFRP coupons, along with a rigorous statistical analysis of test results.

4.3.1 Results of Control Tension Specimens

The control specimens were tested to establish the baseline mechanical properties to compare with subsequent aged test results. Five R-GFRP and five C-GFRP coupons were tested and the resulting average tensile strengths were 352 and 327 MPa, respectively, and the average Young’s moduli were 29.5 and 18.9 GPa, respectively (Table 4-2).
Figure 4-4(a) shows the control (un-aged) stress-strain curves of the R-GFRP and C-GFRP coupons. All subsequent aged coupons were compared to the control ones and results were expressed in terms of percent retention of mechanical properties. It should be noted that, the extensometer was used on each of the five specimens and that the last point was obtained by linear extrapolation.

For the control post-cured specimens, the average tensile strengths of the R-GFRP and C-GFRP were 359 and 333 MPa, respectively, while the average Young’s moduli were 30.5 and 18.8 GPa, respectively (Table 4-2). This indicates a very modest increase of only 1.9% in the tensile strength for the post-cured coupons compared to the control samples. These results are assuring in that the aging at 55°C will note lead to any significant post-curing, and as such, strength retentions can be established by directly comparing results to control specimens. It should be noted here that both GFRP products were fabricated by pultrusion. In this process, the material is cured at high temperature while being pulled through a die. This explains the insignificant increase in strength of the post-cured specimens in this case, as opposed to GFRP made by the wet layup process and used for rehabilitation of structures. The latter is cured at room temperature, and hence post-curing could lead to significant increase in tensile strength.

4.3.2 Results of Environmentally Aged Tension Specimens

Table 4-2 shows a clear reduction in tensile strength of the aged specimens over time. This reduction also becomes larger with higher temperatures. Figures 4-4(b) and (c) show the stress-strain curves of aged specimens at the highest temperature of 55°C, after 150
and 224 days, respectively. Figures 4-5(a) and (b) compare the tensile strengths of the R-GFRP and C-GFRP, respectively, of the aged specimens at various time intervals and temperatures, with control specimens. The standard deviation was calculated for each set and is represented by the error bars.

**Behavior at Various Temperatures:** The R-GFRP did not experience a steady reduction over the exposure period in the 23°C tank (Fig. 4-5(a)). No reduction occurred at 14 days, then a period of steady reduction followed, until 97 days, when a pronounced reduction occurred within a short time. The total reduction at 224 days was 24%. In contrast, the C-GFRP experienced a steady reduction in strength over the exposure period, reaching a total reduction of 22% after 224 days (Fig. 4-5(b)). Unlike the room temperature tank, in the 40°C tank, the R-GFRP showed some reduction at 14 days but also experienced an abrupt reduction in strength between 97 and 150 days in (Fig. 4-5(a)). At 224 days, it reached a total reduction in strength of 28%. The C-GFRP had a steady reduction, reaching 29% at 224 days (Fig. 4-5(b)). The exposure at 55°C certainly resulted in the most reductions. The R-GFRP again showed the abrupt reduction between 97 and at the 150 days and reached the maximum reduction of 36% at 224 days (Fig. 4-5(a)). The C-GFRP again showed steady reduction reaching a maximum of 38% at 224 days (Fig. 4-5(b)).

**Tensile Strength Retention:** Figure 4-6 shows the variation of tensile strength retentions with time. Generally speaking, the R-GFRP and C-GFRP showed comparable degradation at the end of the 224 days. The tensile strength retentions of the R-GFRP at the 23°C, 40°C, and 55°C were 76%, 72%, and 64%, respectively, while for the C-GFRP were 78%, 71%, and 62%, respectively. It has been established earlier that insignificant
post-curing occurs in these types of GFRP and therefore the reported strength retentions, relative to control specimens, are reasonably accurate.

**Failure Modes:** The failure mode of the coupons was also assessed as per ASTM D3039 (2008) standard. All control and aged R-GFRP and C-GFRP coupons experienced a very consistent LGM (Lateral Gauge Middle) failure type, where fibres fractured at a section near the mid-length of the coupon, as shown in Fig. 4-7.

**Elastic Modulus:** The moduli for the control R-GFRP and C-GFRP are 29.5 and 18.9 GPa, respectively. Figure 4-8 shows the variation of elastic modulus with time for the R-GFRP and C-GFRP coupons, at the three different temperatures, compared to control specimens. The elastic modulus of each coupon was found from the slope of the stress-strain curve. The values for each set of five coupons were averaged and plotted in Fig. 4-8. The standard deviation was calculated for each set and is represented by the error bars. Figure 4-8 clearly shows very little reduction in elastic modulus in both types of GFRP. The maximum reductions were 11% for both, occurring at 224 days in 40°C for R-GFRP and at 97 days in 40°C for C-GFRP.

**4.3.3 Results of Microstructure Analysis Tests**

FTIR analysis of control (unconditioned) and aged R-GFRP and C-GFRP samples was conducted on the surface of the samples. This analysis was performed to determine if hydrolysis reactions occurred in the polymer resin, which can lead to an important loss of mechanical properties. When a hydrolysis reaction occurs, new hydroxyl groups are formed and the corresponding infrared band increases. Changes in the peak intensity were quantified by determining the ratio of the OH peak to the resin’s carbon–hydrogen
The stretching peak, which is not affected by conditioning. The experimental ratios for the aged R-GFRP and C-GFRP polyester-based samples immersed in salt solution at 55°C were 0.91 and 0.87, respectively, compared to 0.48 and 0.42 for unconditioned samples, respectively. These results lead to the conclusion that chemical degradation of the polymer occurred at the surface of the polyester-based samples, which is in direct contact with the solution during the immersion. This was confirmed by micrographs obtained using SEM of external surfaces of control and aged R-GFRP and C-GFRP samples (Fig. 4-9). It can be observed that some cracks have appeared due to the hydrolysis of the polymer matrix which could lead to an increase of the penetration of the surrounding solution into the composite and a potential decrease of the mechanical strengths. This observation explains the losses in mechanical properties.

No significant losses of T_g were measured by DSC as the values for the control and aged R-GFRP samples were 109°C and 108°C, respectively, and for the C-GFRP samples were 104°C and 103°C, respectively. These results lead to the conclusion that the polymer matrix is not fully degraded by the hydrolysis and corroborate the fact that hydrolysis is a time-dependent mechanism.

Figure 4-10 compares cross-sections of control and aged samples of R-GFRP and C-GFRP. The obtained micrographs revealed fairly intact interfaces between glass fibres and polyester resin in control samples for both types of GFRP (Figs. 4-10(a and c)). However, after long-term conditioning, the interfaces show that some parts are susceptible to deterioration. The micrographs of Figs. 4-10(b and d) clearly show some fibres partially or completely debonded from the polymer matrix for R-GFRP and C-GFRP samples, respectively. The significant decrease in tensile strengths can be due to
this deterioration of the interface between the fibres and resin which becomes significant by the end of the aging period.

4.3.4 Statistical Assessment Using Analysis of Variance (ANOVA)
The ANOVA technique was used to assess the level of variability and its significance for the various parameters of the study by comparing different sets of coupons that reflect a particular parameter. The null hypothesis was that the results from the different sets were from the same statistical population. As such, if the null hypothesis is “accepted”, no significant difference is observed between the sets being compared. Table 4-3 provides a summary of the ANOVA analysis and outcomes. In this table, MSW is the mean square within the group, MSB is the mean square between groups, SSW is the sum of squares within the group, SSB is the sum of squares between groups, dF is the total degrees of freedom used to calculate MSW and MSB, and F_{crit} (or F_{0.05}) is the critical value of F (taken at 95% confidence level in this case) (Chase and Brown, 1997)). If F is smaller than F_{crit}, then the null hypothesis is “accepted”, suggesting no significance difference between the two groups of data.

Four different parameters were assessed, namely, age effect on strength, temperature effect on strength, combined age and temperature effect on Young’s modulus, and the effect of GFRP type on strength retention. The first parameter was the effect of the 224 days submersion of the R-GFRP and C-GFRP coupons in the tanks at the three different temperatures, including six sets. As shown in Table 3, the reductions in strength after the 224 days, relative to the control cases, are proven to be ‘Significant’. For the second parameter, the temperature effect on tensile strength between various sets
after 224 days, included six sets as shown in Table 3. In both R-GFRP and C-GFRP, the differences in strengths between the 23°C and 55°C sets and between the 40°C and 55°C sets are proven to be ‘Significant’, while the differences between the 23°C and 40°C sets are ‘Not’. For the third parameter, the sets with the maximum differences in Young’s modulus relative to the control, which were R-40-224 and C-40-87 with 11% reduction, were selected for the assessment. Table 3 shows that in both cases the difference is ‘Significant’. Finally, the fourth parameter compares the strength retention of the R-GFRP and C-GFRP at the three different temperatures after 224 days, including three sets. Table 4-3 shows that the difference between the two types is ‘Not’ considered significant after 224 days.

4.3.5 Prediction of Long-Term Behavior using Arrhenius Relationship

For a certain location’s annual mean temperature, the strength retention of the composite can be predicted over its service life by using the Arrhenius model. In this study, the reductions in strength of the R-GFRP and C-GFRP are predicted over their service life at three mean annual temperatures, namely 3°C, 10°C and 20°C, representing central Canada, southern parts of Canada and southern parts of the United States, respectively. The linearized form of the Arrhenius equation can be written as follows (Meyer et al. 1994):

\[
\ln\left(\frac{1}{k}\right) = \frac{E_a}{R \cdot T} - \ln(A)
\]  

(4-1)

where \(k\)=degradation rate (1/time); \(A\)=constant relative to the material and degradation process; \(E_a\)=activation energy; \(R\)=universal gas constant; \(T\)=Temperature (Kelvin). \(\ln(A)\) is the intercept of the regression line with the y-axis and \(E_a/R\) is the slope in the Arrhenius plot. As such, \(E_a\) and \(A\) can be determined once Arrhenius model is plotted. A number of assumptions are made, namely, only one chemical degradation mode can
dominate over the aging process and this mode cannot change with time or temperature, and the FRP must be aged in an aqueous solution, not a dry environment (Bank et al. 2003). At least three elevated temperatures are necessary to get accurate results from the Arrhenius model (Gerritse, 1998).

The procedure used to determine the long-term strength retention of the GFRPs was outlined by Bank et al. (2003). The first step is to produce the relationship between property retention, expressed in percentage of the original strength, and the natural logarithm of time for given exposure temperatures, as shown in Fig. 4-11. Each data set is fit to a trend line and equations are produced as also shown in the figure. It can be seen that the trend lines have $R^2$ values well above the minimum of 0.8 recommended by Bank et al. (2003). The equations of the regression lines are used to produce the Arrhenius plots (Fig. 4-12), which describe the relationship of the natural logarithm of time over the inverse of temperature, expressed in 1000/K for given property retentions. This graph is then used to predict the property retention over the composite’s service life at selected temperatures lower than those used in the experiments. Predictions of tensile strength retentions of R-GFRP and C-GFRP immersed in saltwater at site temperatures of $3^o$, $10^o$, $20^o$C are shown in Fig. 4-13. The following Equations represent the strength retention over service life for the R-GFRP and C-GFRP:

\[
\left( \frac{f_u(t)}{f_u(t=0)} \right) \times 100 = -7.07 \ln(t) + 77.3 \quad \text{(R-GFRP at } T = 3^o\text{C)} \quad (4-2)
\]

\[
\left( \frac{f_u(t)}{f_u(t=0)} \right) \times 100 = -7.36 \ln(t) + 75.5 \quad \text{(R-GFRP at } T = 10^o\text{C)} \quad (4-3)
\]

\[
\left( \frac{f_u(t)}{f_u(t=0)} \right) \times 100 = -7.79 \ln(t) + 72.9 \quad \text{(R-GFRP at } T = 20^o\text{C)} \quad (4-4)
\]
where \( f_u(t) \) is the tensile strength at a given time \( t \), where \( t \) is in years and \( f_u(t=0) \) is the short term tensile strength. After 100 years in service, immersed in saltwater at 3\(^\circ\)C, 10\(^\circ\)C and 20\(^\circ\)C, the model suggests that the tensile strength retentions of the R-GFRP are estimated to be 45, 42 and 37\%, respectively, and for the C-GFRP are estimated to be 65, 61 and 55\%, respectively. It is interesting to note that the tensile strength retentions of the C-GFRP are higher than the R-GFRP, despite the higher strength and stiffness of the control R-GFRP compared to C-GFRP. Also, results clearly show that the majority of the reduction occurs early in service life, regardless of site temperature. For example, the C-GFRP would lose about 24-30\% from its original strength after only 5 years immersion in saltwater, followed by another 8-11\% reduction beyond the first 5 years up to 50 years. Then, minimal reductions of 5-7\% occur during the following 150-years.

4.4 SUMMARY
This study addressed the durability of glass-fibre reinforced polymer (GFRP) pultruded structural sections used in bridge deck applications, namely, a flat plate with T-shape ribs (R-GFRP) and a corrugated plate (C-GFRP). Standard coupons were aged for up to 224 days at 23, 40 and 55\(^\circ\)C in separate baths of 3\% salt solutions simulating de-icing conditions. The tensile strength retentions and Young’s moduli were measured periodically. Data were assessed using Analysis of variance (ANOVA). Micro-structure
assessments using Differential Scanning Calorimetry, Fourier Transform Infrared Spectroscopy and Scanning Electron Microscopy were carried out to provide additional assessment of degradation. It was shown that after 224 days, the tensile strength retentions of the R- and C-GFRPs were similar, and reduced from 77 to 63% as the temperature increased from 23 to 55°C. The observed reductions were confirmed by micro-graphs showing some surface cracks and separations between fibres and matrix, but results also showed that the polymer matrix is not fully degraded by the hydrolysis as no significant changes occurred in the glass transition temperature after exposure. When data was fitted in the Arrhenius service life model, it showed that after 100 years, R-GFRP will suffer more deterioration than C-GFRP as the strength retentions at a location with annual mean temperatures of 10°C were 42 and 61%, respectively.
Table 4-1: Test Matrix

<table>
<thead>
<tr>
<th>Type</th>
<th>Number of coupons</th>
<th>Environmental condition</th>
<th>Temp. (°C)</th>
<th>Time in tanks (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ribbed (R-GFRP)</td>
<td>5</td>
<td>Control, dry</td>
<td>23</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td>23</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Immersed in 3% salt water</td>
<td></td>
<td>224</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td>40</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>224</td>
</tr>
<tr>
<td>Corrugated (C-GFRP)</td>
<td>5</td>
<td>Control, dry</td>
<td>23</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td>23</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Immersed in 3% salt water</td>
<td></td>
<td>224</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td>40</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>224</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td>55</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td></td>
<td></td>
<td>224</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td></td>
<td></td>
<td>224</td>
</tr>
</tbody>
</table>
Table 4-2: Summary of test results

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Type</th>
<th>No of coupons to calculate mean</th>
<th>Mean maximum strength (MPa)</th>
<th>Standard deviation (MPa)</th>
<th>Reduction in strength (%)</th>
<th>Mean Young’s modulus (GPa)</th>
<th>Standard deviation (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-Control</td>
<td></td>
<td>5</td>
<td>352</td>
<td>16.5</td>
<td>-</td>
<td>29.5</td>
<td>2.4</td>
</tr>
<tr>
<td>R-23-14</td>
<td></td>
<td>5</td>
<td>356</td>
<td>10.3</td>
<td>+1.1</td>
<td>28.7</td>
<td>1.5</td>
</tr>
<tr>
<td>R-23-33</td>
<td></td>
<td>5</td>
<td>334</td>
<td>32.5</td>
<td>-5.2</td>
<td>29.6</td>
<td>1.1</td>
</tr>
<tr>
<td>R-23-97</td>
<td></td>
<td>5</td>
<td>315</td>
<td>10.6</td>
<td>-10.7</td>
<td>27.8</td>
<td>1.8</td>
</tr>
<tr>
<td>R-23-150</td>
<td></td>
<td>5</td>
<td>274</td>
<td>11.1</td>
<td>-22.1</td>
<td>28.3</td>
<td>2.0</td>
</tr>
<tr>
<td>R-23-224</td>
<td></td>
<td>5</td>
<td>268</td>
<td>20.5</td>
<td>-23.9</td>
<td>30.2</td>
<td>1.3</td>
</tr>
<tr>
<td>R-40-14</td>
<td>Ribbed R-GFRP</td>
<td>5</td>
<td>340</td>
<td>20.3</td>
<td>-3.5</td>
<td>29.9</td>
<td>2.2</td>
</tr>
<tr>
<td>R-40-33</td>
<td></td>
<td>5</td>
<td>320</td>
<td>13.4</td>
<td>-9.1</td>
<td>28.3</td>
<td>1.6</td>
</tr>
<tr>
<td>R-40-97</td>
<td></td>
<td>5</td>
<td>300</td>
<td>20.6</td>
<td>-14.8</td>
<td>27.3</td>
<td>1.8</td>
</tr>
<tr>
<td>R-40-150</td>
<td></td>
<td>5</td>
<td>269</td>
<td>25.8</td>
<td>-23.8</td>
<td>26.8</td>
<td>1.0</td>
</tr>
<tr>
<td>R-40-224</td>
<td></td>
<td>5</td>
<td>254</td>
<td>8.3</td>
<td>-28.0</td>
<td>26.4</td>
<td>1.3</td>
</tr>
<tr>
<td>R-55-14</td>
<td></td>
<td>5</td>
<td>329</td>
<td>8.9</td>
<td>-6.5</td>
<td>28.8</td>
<td>2.7</td>
</tr>
<tr>
<td>R-55-33</td>
<td></td>
<td>5</td>
<td>312</td>
<td>29.1</td>
<td>-11.5</td>
<td>29.1</td>
<td>2.8</td>
</tr>
<tr>
<td>R-55-97</td>
<td></td>
<td>5</td>
<td>294</td>
<td>16.5</td>
<td>-16.6</td>
<td>26.6</td>
<td>1.3</td>
</tr>
<tr>
<td>R-55-150</td>
<td></td>
<td>5</td>
<td>235</td>
<td>24.5</td>
<td>-33.2</td>
<td>27.5</td>
<td>1.1</td>
</tr>
<tr>
<td>R-55-224</td>
<td></td>
<td>4</td>
<td>227</td>
<td>12.9</td>
<td>-35.6</td>
<td>27.5</td>
<td>3.2</td>
</tr>
<tr>
<td>Post-cured R-GFRP</td>
<td></td>
<td>5</td>
<td>359</td>
<td>19.6</td>
<td>+1.9</td>
<td>30.5</td>
<td>2.8</td>
</tr>
<tr>
<td>C-Control</td>
<td></td>
<td>5</td>
<td>327</td>
<td>14.9</td>
<td>-</td>
<td>18.9</td>
<td>1.1</td>
</tr>
<tr>
<td>C-23-14</td>
<td></td>
<td>5</td>
<td>308</td>
<td>17.5</td>
<td>-5.9</td>
<td>17.6</td>
<td>0.9</td>
</tr>
<tr>
<td>C-23-33</td>
<td></td>
<td>5</td>
<td>284</td>
<td>14.6</td>
<td>-13.0</td>
<td>17.7</td>
<td>1.1</td>
</tr>
<tr>
<td>C-23-97</td>
<td></td>
<td>5</td>
<td>272</td>
<td>26.5</td>
<td>-16.7</td>
<td>17.0</td>
<td>1.2</td>
</tr>
<tr>
<td>C-23-150</td>
<td></td>
<td>5</td>
<td>269</td>
<td>28.3</td>
<td>-17.8</td>
<td>18.0</td>
<td>0.6</td>
</tr>
<tr>
<td>C-23-224</td>
<td></td>
<td>5</td>
<td>254</td>
<td>16.1</td>
<td>-22.4</td>
<td>17.4</td>
<td>0.4</td>
</tr>
<tr>
<td>C-40-14</td>
<td>Corrugated C-GFRP</td>
<td>5</td>
<td>294</td>
<td>22.8</td>
<td>-10.0</td>
<td>17.8</td>
<td>1.0</td>
</tr>
<tr>
<td>C-40-33</td>
<td></td>
<td>5</td>
<td>280</td>
<td>12.7</td>
<td>-14.5</td>
<td>17.3</td>
<td>1.4</td>
</tr>
<tr>
<td>C-40-97</td>
<td></td>
<td>5</td>
<td>268</td>
<td>15.1</td>
<td>-18.0</td>
<td>16.8</td>
<td>1.5</td>
</tr>
<tr>
<td>C-40-150</td>
<td></td>
<td>5</td>
<td>249</td>
<td>34.9</td>
<td>-23.8</td>
<td>17.7</td>
<td>0.8</td>
</tr>
<tr>
<td>C-40-224</td>
<td></td>
<td>5</td>
<td>233</td>
<td>14.1</td>
<td>-28.9</td>
<td>17.0</td>
<td>0.6</td>
</tr>
<tr>
<td>C-55-14</td>
<td></td>
<td>5</td>
<td>290</td>
<td>32.4</td>
<td>-11.4</td>
<td>17.0</td>
<td>1.7</td>
</tr>
<tr>
<td>C-55-33</td>
<td></td>
<td>5</td>
<td>252</td>
<td>26.9</td>
<td>-23.0</td>
<td>17.4</td>
<td>0.9</td>
</tr>
<tr>
<td>C-55-97</td>
<td></td>
<td>5</td>
<td>211</td>
<td>11.0</td>
<td>-35.4</td>
<td>16.9</td>
<td>1.2</td>
</tr>
<tr>
<td>C-55-150</td>
<td></td>
<td>5</td>
<td>205</td>
<td>16.5</td>
<td>-37.2</td>
<td>18.4</td>
<td>0.9</td>
</tr>
<tr>
<td>C-55-224</td>
<td></td>
<td>4</td>
<td>202</td>
<td>12.6</td>
<td>-38.3</td>
<td>17.4</td>
<td>0.6</td>
</tr>
<tr>
<td>Post-cured C-GFRP</td>
<td></td>
<td>5</td>
<td>333</td>
<td>16.8</td>
<td>+1.9</td>
<td>18.8</td>
<td>1.2</td>
</tr>
</tbody>
</table>
Table 4-3: Results of ANOVA statistical analysis

| Sample groups       | Parameter                        | Sum     | Average | Variance | SSB | SSW | MSB | MSW | df | F   | F_α   | Results | Conclusions |
|---------------------|----------------------------------|---------|---------|----------|-----|-----|-----|-----|----|-----|-------|--------|-------------|-------------|
| (R-Control, R-23-224) | Age effect on strength           | (1761, 1341) | (352, 268) | (133, 418) | 17649 | 2205 | 17649 | 276 | (1.8) | 64 | 5.3   | Reject | Significant |
| (R-Control, R-40-224) |                                  | (1761, 1268) | (352, 254) | (133, 69) | 24349 | 888  | 24349 | 101 | (1.8) | 241 | 5.3   | Reject | Significant |
| (R-Control, R-55-224) |                                  | (1761, 987) | (352, 227) | (133, 166) | 35912 | 1031 | 35912 | 147 | (1.7) | 93.3 | 5.3   | Reject | Significant |
| (C-Control, C-23-224) |                                  | (1634, 1268) | (327, 254) | (274, 269) | 13449 | 2134 | 13449 | 267 | (1.8) | 238 | 5.6   | Reject | Significant |
| (C-Control, C-40-224) |                                  | (1634, 1163) | (327, 233) | (274, 198) | 22259 | 1886 | 22259 | 236 | (1.8) | 94  | 5.3   | Reject | Significant |
| (C-Control, C-55-224) |                                  | (1634, 762) | (327, 198) | (274, 105) | 30663 | 1809 | 30663 | 201 | (1.7) | 184 | 5.6   | Reject | Significant |
| (R-23-224, R-55-224) | Temperature effect on strength   | (1341, 987) | (268, 227) | (418, 166) | 3827 | 2172 | 3827 | 310 | (1.7) | 12.33 | 5.6   | Reject | Significant |
| (R-23-224, R-40-224) |                                  | (1341, 1268) | (268, 254) | (418, 69) | 538  | 1949 | 638  | 244 | (1.8) | 2.2  | 5.3   | Accept | NOT       |
| (R-40-224, R-55-224) |                                  | (1268, 987) | (254, 227) | (69, 166) | 1660 | 776  | 1660 | 111 | (1.7) | 14.45 | 5.6   | Reject | Significant |
| (C-23-224, C-55-224) |                                  | (1629, 896) | (284, 292) | (269, 158) | 6902 | 1514 | 6902 | 216 | (1.7) | 27.8 | 5.6   | Reject | Significant |
| (C-23-224, C-40-224) |                                  | (1629, 1163) | (254, 233) | (269, 198) | 1104 | 1830 | 1104 | 229 | (1.8) | 4.83 | 5.3   | Accept | NOT       |
| (C-40-224, C-55-224) |                                  | (1613, 806) | (233, 292) | (198, 158) | 2130 | 1266 | 2130 | 181 | (1.7) | 11.8 | 5.6   | Reject | Significant |
| (R-Control, R-40-224) | Age and Temp. effects on Modulus | (148, 132) | (30, 26)  | (5.5, 16) | 25   | 29   | 25   | 3.6 | (1.8) | 0.07 | 5.3   | Reject | Significant |
| (C-Control, C-40-97)  |                                  | (96, 84)   | (19, 17)  | (0.8, 2.3) | 16   | 12   | 16   | 1.5 | (1.5) | 10.3 | 5.3   | Reject | Significant |
| (R-23-224, C-23-224) | Effect of FRP type on strength   | (361, 388) | (76, 78)  | (34, 24)  | 5    | 232  | 5    | 29  | (1.8) | 0.17 | 5.3   | Accept | NOT       |
| (R-40-224, C-40-224) |                                  | (360, 366) | (72, 71)  | (5.6, 10.4) | 1.8 | 96   | 1.8  | 12  | (1.8) | 0.15 | 5.3   | Accept | NOT       |
| (R-55-224, C-55-224) |                                  | (257, 247) | (83, 62)  | (13.4, 14.8) | 15 | 85   | 15   | 14  | (1.6) | 1.04 | 6     | Accept | NOT       |
Figure 4-1: Various types of pultruded GFRP sections used in bridge decks: (a and b) are stay-in-place structural forms, and (c) is an all-FRP deck.

Figure 4-2: Schematic view of coupons cut in the longitudinal direction from the GFRP panels.
Figure 4-3: Environmental tank setup; and b) Tension test setup
Figure 4-4: Stress-strain curves for R-GFRP and C-GFRP for: a) control; b) after 150 days at 55°C; and c) after 224 days at 55°C (the shown line presents an average value for five samples)
Figure 4-5: Tensile strength of: a) R-GFRP; and b) C-GFRP aged in saltwater at 23°C, 40°C, and 55°C

Figure 4-6: Tensile strength retention for R-GFRP and C-GFRP over 224 days at various temperatures
Figure 4-7: Failure modes

Figure 4-8: Elastic moduli of: a) R-GFRP; and b) C-GFRP aged in saltwater at 23°C, 40°C, and 55°C
Figure 4-9: Micrographs of the external surface for: a) R-GFRP reference sample, b) R-GFRP sample aged in salt solution for 204 days at 55°C, c) C-GFRP reference sample, and, d) C-GFRP sample aged in salt solution for 214 days at 55°C.
Figure 4-10: Micrographs at the fibre/matrix interface before mechanical tests for: 
a) R-GFRP reference sample, b) R-GFRP sample aged in salt solution for 204 days at 55°C, c) C-GFRP reference sample, and, d) C-GFRP sample aged in salt solution for 214 days at 55°C.
Figure 4-11: Variation of tensile strength retention with time logarithmic for: a) R-GFRP and b) C-GFRP
Figure 4-12: Arrhenius plots for: a) R-GFRP and b) C-GFRP
Figure 4-13: Predicted relationship between the retained tensile strength and service life at annual mean temperatures of 3°C, 10°C, and 20°C for: a) R-GFRP and b) C-GFRP
Chapter 5: Conclusions

5.1 Durability of Bridge Deck with FRP Stay-in-Place Structural Forms under Freeze-Thaw Cycles

The effect of aggressive freeze-thaw (FT) cycles on the strength of concrete bridge decks built using GFRP stay-in-place (SIP) structural forms that replace all bottom rebar mesh has been studied. Up to 300 FT cycles were carried out at a concrete core temperature range of +5 °C to -18 °C, which represents an air temperature range of +13 °C to -25 °C. The study addressed various surface treatments and bond conditions of the GFRP forms and their lap splices. Freezing was in air while thawing was by water. Conditions during thawing, whether by submersion or without submersion, were compared and the effect of perforating the GFRP forms was studied. The following conclusions can be drawn:

1. After 100, 200 and 300 FT cycles, where thawing was by submersion in water, the ultimate strength of the decks changed by -9%, -8%, and +3 to +6%, respectively. Given that identical repetitions varied by 5%, it is concluded that specimens survived the FT exposure, without any noticeable reduction in strength, even for deck with unbonded forms and unbonded splice.

2. At the end of 300 FT cycles, decks subjected to thawing without submersion showed 11% lower strength than those submerged during thawing, and were 8% lower strength than control specimens. Submersion has caused further curing of concrete, increasing its strength.

3. The 300 FT cycles resulted in 23% reduction in tensile strength, and 11% in Young’s modulus, of the GFRP SIP form on its own. However, this reduction
was not manifested as weakening of the overall deck capacity, as the failure mode was not governed by the form.

4. Perforating the FRP SIP forms, to drain any entrapped water at concrete-form interface, proved unnecessary, as the strengths of decks with perforated and unperforated forms were identical after the 300 FT cycles.

5. The deck without any surface treatment or bond between concrete and GFRP SIP form and without bonded lap splice represents the simplest and easiest construction. It showed 21% lower capacity than those with bonded forms and bonded splices; nonetheless, this lower capacity is still 3.9 higher than the equivalent service load at this scale.

6. All specimens of various surface treatments with bonded or unbonded lap splices and those exposed to FT cycles failed similarly, by concrete punching shear. At failure, the tensile stress in the GFRP SIP forms and top GFRP rebar did not exceed 15-20% of its ultimate values, while concrete compressive strain did not exceed 70% of crushing strain.

5.2 The Effect of the Shape of FRP Stay-in-Place Structural Form on Durability of Concrete Members under Freeze-Thaw Cycles

This study examined the effect of 300 freeze-thaw cycles at a temperature range of +5 °C to -18 °C on two types of GFRP stay-in-place structural forms for concrete structures, namely a flat plate with T-shape ribs and a corrugated plate. Different surface preparation techniques were used for the forms. The following conclusions are drawn:

1. In specimens with flat forms having T-shape ribs, freeze-thaw exposure did
not cause any strength reduction or change in failure modes, neither in the one without surface treatment, nor in the one with bonded aggregates. This suggests that the embedment of the T-shape ribs in concrete counteracts any ‘frost-jacking’ effect between the flat plate and concrete.

2. In specimens with corrugated forms, freeze-thaw exposure resulted in 21% and 18% reductions in strength of specimens with wet adhesive and bonded aggregates, respectively. Also, failure mode changed from concrete shear to bond failure. Unlike ribbed panels, this system has no mechanical interlocking between the form and concrete.

3. A specimen with untreated corrugated form showed a strength that is only 21% and 26% of those treated with wet adhesive and bonded aggregates, respectively. For flat plate forms with T-shape ribs, the specimen with untreated form had 44% of the strength of that with bonded aggregates.

4. Specimens with untreated flat plates with T-shape ribs showed a remarkable behavior characterized by pseudo-ductility arising from gradual slip and eventually shearing off, of the rib from the base. When bonded aggregate treatment was used, concrete shear failure occurred at higher strength with no pseudo-ductility.

5.3 Durability Study on Pultruded FRP sections used in Bridge Deck Applications
The durability of two pultruded GFRP sections, namely a flat plate with T-shape ribs (R-GFRP) and a corrugated plate (C-GFRP), both suited for bridge deck applications, was assessed. Coupons were cut and aged for up to 224 days in a 3% salt-solution at three
different temperatures. The tensile strength retentions and Young’s modulus were then assessed periodically and a rigorous statistical assessment using Analysis of Variance (ANOVA) was carried out. The Arrhenius model was used to establish service life strength retentions. Also, micro-structure assessments using Differential Scanning Calorimetry (DSC), Fourier Transform Infrared Spectroscopy (FTIR) and Scanning Electron Microscopy (SEM) were carried out. The following conclusions are drawn:

1. After 224 days of aging, the tensile strength retentions of the R-GFRP at 23, 40, and 55°C were 76, 72 and 64, respectively, and for the C-GFRP were 78, 71 and 62%, respectively.

2. It follows from previous point that R-GFRP and C-GFRP did not show a distinct difference in degradations after 224 days. However, when data is fitted in the Arrhenius model, it suggests that after 100 years, R-GFRP will sufferer more deterioration than C-GFRP. The estimated strength retentions at locations with annual mean temperatures of 3, 10 and 20°C were 45, 42 and 37%, respectively, for R-GFRP and 65, 61 and 55% for C-GFRP. The majority of losses occur in the first five years.

3. While the C-GFRP showed a steady and gradual reduction in strength over 224 days, the R-GFRP showed steady reduction up to 97 days, then appear to have experienced a rapid decline between 97 and 150 days.

4. The Young’s moduli of both types of GFRP had little reduction of about 11% during the 224 days of exposure but generally appeared stable.

5. All control and aged coupons of both GFRP types experienced a very consistent LGM (Lateral Gage Middle) failure mode, where fibres fractured at a section near mid-length.
6. Neither type of GFRP showed evidence of post-curing when heated to 90°C for 48 hours. This may be attributed to the process of pultrusion and curing using heated dies.

7. FTIR tests revealed a chemical degradation of the polyester polymer of both GFRP types at the surfaces which are in direct contact with the solution. This was further confirmed by SEM images showing surface cracks, which explains the losses in mechanical properties.

8. DSC test results showed that the polymer matrix is not fully degraded by the hydrolysis as no significant changes occurred in the glass transition temperature (T_g) of either type of GFRP after exposure. T_g values of control and aged R-GFRP were 109 and 108°C, respectively, while those of C-GFRP were 104 and 103°C, respectively.

9. SEM micrographs revealed distinct debonding at the interface between the fibres and matrix in both types of GFRP after aging. The significant decreases in tensile strengths can be due to this deterioration of the interface.
REFERENCES


Properties of GFRP made of Furfuryl Alcohol Bio-Resin Compared to Epoxy”,

Stay-in-Place Structural Forms with Interlocking Connections”, ASCE Journal of
Composites for Construction, 16(1):110-117.

concrete structures." Proc., Durability of Fiber Reinforced Polymers (FRP)

durability between fibre reinforced polymer plate reinforcement and concrete."

behaviour of reinforced concrete beams strengthened in flexure with fibre
reinforced polymer sheets.” Canadian Journal of Civil Engineering 30(6): 1081-
1088.

reinforced concrete beams strengthened by fibre reinforced sheets". Canadian
Society for Civil Engineering, Montréal, Que.

permanent formwork for concrete members.” Journal of Composites for
Construction, 2(2):78–86.


Concrete and Subjected to Salt Solution,“ J. Compos. Constr., 16(2):217-224.


