EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS OF
CONCRETE BRIDGE DECKS WITH STRUCTURAL FRP STAY-IN-PLACE FORMS

by

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Abstract

Stay-In-Place (SIP) formwork systems are widely used for concrete slabs in industry due to their relative ease and speed of construction. Conventionally, corrugated metal sheets or precast panels are used as formwork. In recent years, the SIP formwork technique has been proposed in conjunction with Fiber Reinforced Polymer (FRP) composites. The resulting system combines the construction advantages of SIP formwork with the durability and corrosion resistance of FRP materials. Bridge decks are a particularly enticing application due to their exposure to harsh environmental conditions and the need for rapid construction to minimize traffic disruptions. This study broadly evaluates FRP SIP formwork for concrete bridge decks both experimentally and numerically. In total, 9 full scale bridge deck sections, 32 small scale decks and more than 40 auxiliary tests were conducted, including the construction and testing of a full bridge at scale. Additionally, a numerical model was developed to predict punching shear failure based on the theory of plates and shells. Experimental testing was conducted on two FRP SIP form configurations, namely flat plates with T-shape stiffeners and corrugated plates, and used a variety of different detailing and geometries. Some of the investigated parameters included the width effect of bridge deck section tests, the effect of deck span, the effect of bond at the FRP-concrete interface, the panel-to-panel splice configuration, concrete strength, and boundary condition at support, including a monolithic connection with precise girders. Results of the study include the determination of a critical aspect ratio for bridge deck sections, optimization of the panel-to-panel splice detail, and an assessment of the in-plane restraint available to interior span bridge decks. The numerical model, based on the Levy solution for loaded plates, produces a flexural response for a variety of bridge deck configurations and geometries. A failure criterion was applied to establish the punching shear capacity. The model was evaluated against experimental results and provided good correlation. It was then used to investigate a variety of FRP plate thicknesses, spans and effective widths for full scale FRP SIP formwork bridge decks.
Acknowledgements

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Statement of Originality

I hereby certify that all of the work described within this thesis is the original work of the author. Any published (or unpublished) ideas and/or techniques from the work of others are fully acknowledged in accordance with the standard referencing practices.

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Chapter 1

Introduction

1.1 General
Reinforced concrete structures are generally constructed using temporary formwork to accommodate fresh concrete. Once the concrete has set, the forms are removed and either disposed of or reused. An alternative to this method is the use of Stay-in-Place (SIP) forms, also known as permanent or lost forms. Instead of being removed after setting, these forms become a part of the permanent structure. These SIP forms are most commonly either corrugated cold rolled steel or precast concrete. They may be designed as a structural component of the final structure (as with structural SIP forms) or simply as a non-structural lost formwork. The primary advantage of SIP form construction is the additional speed and ease of construction which they offer over conventional formwork. These benefits are particularly realised in the construction of slabs which are of regular and repetitive geometry and in situations where removal (or ‘stripping’) of formwork is impractical. For all of the above reasons, the concrete bridge deck of slab-on-girder bridges is an ideal candidate for SIP formwork. Additional optimization can be achieved when the form itself can act as a structural SIP form and reduce or replace the deck’s reinforcing bars.

Fiber Reinforced Polymer composites (FRPs) have been gaining acceptance as a suitable concrete reinforcement for some time. Their inherent electrochemical corrosion resistance makes them particularly well suited for infrastructure applications subjected to harsh environmental exposures. In these applications, reinforced concrete is typically limited in durability by corrosion of internal steel reinforcement. For a number of reasons the bridge deck component of a bridge structure tends to be the most vulnerable to deterioration, to the extent that bridge decks are often replaced while keeping existing superstructure and substructure elements. In response, FRPs have
been investigated and applied extensively as reinforcing bars for concrete bridge decks and barriers, in some cases completely replacing conventional steel reinforcing bars.

The logical extension of these two concepts is to develop a SIP formwork system using FRP materials. This system, called FRP SIP formwork, combines the constructability advantages of SIP formwork with the enhanced durability of FRPs.

The design of FRP SIP formwork systems is influenced heavily by the geometry of the FRP form panels. These panels may consist of a stiffened or corrugated FRP plates supported between parallel girders and cast integrally with the concrete deck. Several field applications have been executed, but in each case a unique form panel was used and laboratory or numerical investigations supported the validation of a particular design. This thesis provides a comprehensive investigation of several configurations of FRP SIP formwork systems.

1.2 Objectives

This thesis investigates the structural performance of FRP SIP form systems for bridge decks. The goals of this thesis can be described by its overarching themes and particular objectives. The following are several major themes of the thesis:

1. Investigate FRP SIP formwork systems from a parametric perspective, rather than a benchmarking approach. Evaluate multiple series of specimens varying single parameters to gain insight on their effect. Develop conclusions which are general in nature to FRP SIP form systems and which can guide the design of future systems.

2. Develop test methods to simulate (as closely as possible) actual in-place deck performance in the laboratory so that lab scale deck sections can be used to develop reliable (as opposed to just conservative) performance data on FRP SIP form systems.

3. Test an FRP SIP form system to failure as part of a full bridge. This is the only method guaranteed to accurately simulate actual as-built boundary conditions. Performance from such tests would effectively set the bar for any bridge deck section test method.
4. Develop an analytical model which is able to predict the punching shear capacity (the universally limiting failure criterion) of FRP SIP form-bridge decks so that it can be used to extend experimental results past the practical limitations of lab testing.

The main aspects of FRP SIP form systems investigated in this thesis are summarized by the following objectives:

5. Assessment of two FRP SIP formwork configurations, namely flat plate with T-shape ribs and corrugated plate, at full scale using bridge deck sections. This includes the development of a test setup which simulates as accurately as possible the boundary conditions typical of concrete bridge decks on precast/prestressed concrete girders. Within the context of these two SIP systems, evaluate critical parameters such as the provision of top reinforcement, monolithic boundary conditions at girders, FRP-concrete interface bond level, and FRP panel thickness and compare the performance of these deck systems to conventional steel-reinforced concrete construction, and to code requirements to determine their suitability for bridge construction.

6. Investigate the limiting attributes of the system in terms of stiffness and ultimate strength and suggest routes for optimized performance based on improving these limitations and relate the performance of FRP SIP formwork systems to hollow ‘all-FRP’ deck systems.

7. Conduct a phase of experimental testing at scale, allowing for greater numbers of specimens and a more thorough analysis of various parameters using a single FRP SIP formwork system. In the process, construct a full slab-on-girder complete bridge at 1:2.75 scale to assess the specific boundary conditions present in full bridge decks This unique specimen can then be used to study a number of parameters including effect of interior/exterior span location, span length, splice detailing, type of construction (RC or FRP SIP form) and load location.
8. Extend the results of these full bridge tests to a series of scaled bridge deck sections investigating, among other things, the effect of concrete strength and deck section width on system performance. Using scale tests, optimize the panel to panel splicing details of this FRP SIP form system by conducting direct tension splice tests, then investigate these optimal splicing details in bridge deck sections.

9. Investigate the effect of panel width (traffic direction) on system performance, with reference to the panel widths typically used in field projects and in combination with optimal splice detailing.

10. Develop an analytical model capable of predicting the full load-deflection response and punching shear capacity of FRP SIP formwork systems based on their geometries and material properties, inclusive of scale, system and boundary conditions. The model can be evaluated against the suite of experimental data developed in the preceding investigations. Finally, a parametric study can be conducted to assess several parameters not investigated experimentally due to practical considerations.

1.3 Scope
This thesis broadly describes the execution and results of an experimental investigation and a numerical model.

The experimental component of this thesis includes both full scale and small scale testing of various bridge deck configurations. At full scale, it includes eight interior bridge deck sections and one cantilever type overhang specimen. The specimens were designed and constructed monolithically on simulated AASHTO type III precast prestressed girders with care taken to simulate accurate boundary conditions. At small scale, a total of 32 decks were tested, including a combination of bridge deck sections and test pads within a full bridge. Where bridge deck sections were used, a test frame was employed to establish boundary conditions similar to either the full scale testing conducted earlier, or the conditions of pads within the full bridge deck. Each
bridge deck section was either cast monolithically with its testing frame, or was grouted and clamped to the testing frame before loading. Reported in this thesis are the various test setups, instrumentation, results and conclusions from this experimental endeavor. With each phase of testing, efforts were made to improve the test method and specimen design based on previous conclusions, with the goal of simulating in-place performance as closely as possible with laboratory results.

An analytical model is developed based on the theory of plates and shells. It includes three essential components. First, the moment/curvature performance of a given FRP SIP form system is developed in the two primary directions both in positive and negative bending. Well established constitutive relationships are employed for concrete and FRP materials present in the cross section. Second, the deflected shape of the slab is calculated for a given load, based on the loading geometry (in this case a pressure applied over a rectangular area), slab geometry and restraints. Three cases of boundary condition are handled: infinitely long panels (effectively as in a full bridge deck), panels of finite width supported at the ends, and panels of finite width unsupported at the ends (as in the case of bridge deck sections). This program iteratively calculates the load-deflection performance of the deck over its entire surface at a resolution determined by the user. Finally, using information from this load-deflection performance, a proposed failure criterion is applied to determine the punching shear load.

1.4 Thesis outline
A brief chapter by chapter outline of the thesis is as follows:

Chapter 2: A manuscript which summarizes the state of the art in FRP SIP formwork bridge decks. Pertinent literature is presented on the topics of experimental testing, analytical modeling, design, field application and further research needs.

Chapter 3: A manuscript which describes the experimental investigation at full scale of a novel FRP SIP formwork system based on a corrugated FRP form pane with interlocking connections.
Chapter 4: A manuscript which describes the experimental investigation at full scale of a second novel FRP SIP form system constructed with ribbed FRP form plates. The plates possess T-shaped ribs which provide mechanical interlock with the concrete infill.

Chapter 5: A manuscript which describes the construction, testing and results of a full slab-on-girder bridge constructed at 1:2.75 scale in the laboratory.

Chapter 6: A manuscript which describes a series of bridge deck section tests at scale, examining various critical parameters including aspect ratio and boundary conditions.

Chapter 7: A manuscript which describes an experimental program aimed at the optimization of the critical panel-to-panel splice detail of the ribbed plate FRP SIP form system.

Chapter 8: A manuscript which describes the development and validation of an analytical model. Validation is conducted using the substantial test data available from earlier testing and a parametric study is conducted.

Chapter 9: General conclusions are drawn from the results of previous chapters and comments are made on the direction of future research and implementation of FRP SIP formwork systems.

References: References are included within each chapter, and may be of different referencing styles depending on journal specific requirements.

At the date of submission of this thesis, seven journal papers representing the seven key chapters, have been submitted for publication. Three have already been published, one has been accepted and three are under review.
Chapter 2

FRP Stay-in-Place structural forms for concrete bridge decks: A state-of-the-art review

2.1 Introduction
Throughout the world, deteriorating infrastructure has created a large demand for the replacement of existing bridges and structures. Bridge decks, in particular, are exposed to harsh environmental conditions. Improvements in their durability will result in lower life cycle costs and less frequent closures for replacements. Fiber Reinforced Polymer (FRP) composites have been widely used for both structural retrofit in the form of thin sheets and plates and new construction in the form of bars and tendons. The concept of using FRP composites for structurally integrated Stay-In-Place (SIP) formwork maximizes the advantages of both FRP and concrete, while simplifying the construction process and reducing construction time. This document addresses this unique FRP application, focusing specifically on the area of bridge decks, where FRP SIP forms have significant potential and great promise. The document combines a state-of-the-art survey of this technology, in terms of research activities and field applications to-date, with some practical considerations, design guidance, and recommendations for further research. The emphasis in this chapter is on FRP SIP form systems that contribute structurally to the deck.

2.1.1 Advantages and limitations
A primary motivation for using FRP composites in construction has been their improved durability in saline environments relative to conventional steel reinforcement. This advantage adds to a number of other important advantages of the SIP form concept. The most common characteristics of FRP SIP form systems for bridge decks are:

(a) The FRP sections act as permanent form for the concrete structure. This characteristic presents cost savings by significantly reducing conventional formwork and by reducing the labor cost and time related to the constructing and stripping of conventional formwork. The total reduction has been reported to be as high as 55% (Berg et al., 2005).

(b) The FRP sections can act as primary bottom reinforcement for the concrete deck where composite action is established between FRP and concrete. This characteristic reduces the amount of conventional reinforcement required by eliminating the bottom reinforcement, and further reduces the time and cost of installing such systems.

(c) Given the opportunity of custom fabrication, FRP SIP form systems lend themselves to optimisation based on material properties of individual components including the type of fibers, fiber orientation, number of layers in the composite section and the geometric properties of the FRP section itself.

(d) The system is versatile in that it can be used for a wide variety of bridge types and girders, including both steel and concrete girder bridges.

As indicated earlier, the inherent durability of FRPs can increase the lifespan of bridge decks relative to their steel reinforced counterparts. This will reduce life cycle cost and frequency of replacement, which are also significant advantages.

FRP SIP systems also have a number of limitations which designers should be aware of—namely:

(a) Using SIP forms pose a problem for bridge inspections as the concrete soffit becomes no longer visible. This drawback is entirely dependent on preferences of engineers and local practices at various Department of Transportation (DOTs).

(b) Water egress is also prevented from the soffit of the slab, although this problem has not been reported in any previous applications of FRP SIP form technology.
(c) High cost of materials may be particularly significant in the case of FRP SIP forms because of their volume and specific configurations. In general, material cost must be weighed against the potentially significant advantages in labor cost reduction and the advantage of rapid construction.
(d) Most contractors in the construction industry are likely to be unfamiliar with this technology. Although the technology is similar to steel SIP formwork, FRP components have certain specific handling and installation requirements.
(e) The designer must take steps to ensure a suitable interface between the FRP panels and the concrete deck in order to establish composite action where a structural contribution is desired. This can be achieved through various means, discussed later in this thesis.

2.2 Research significance
FRP SIP form systems are seeing increasing usage in field projects as test bridges, but several factors currently prevent the system from gathering more widespread application. This document serves as a resource for designers, owners and contractors in presenting information on the design, detailing and cost considerations inherent in FRP SIP form systems. It reviews, in a concise format, the substantial laboratory and analytical investigations that have taken place to-date. Additionally, the document is important for researchers in developing targeted research programs to address the remaining barriers to the further adoption of FRP SIP forms in practice.

2.3 Background
Pultrusion is an economical, continuous method of manufacturing FRP shapes of constant cross-section by pulling a mixture of resin and usually thermosetting resin through a heated die that compacts and cures the material into the desired shape (Strong, 2007). Due to the high initial cost of tooling for pultrusion, most FRP SIP forms are currently manufactured in a limited number of pultruded configurations that were originally designed to serve the structural shapes market. Fig. 1 shows a general layout of the most common sections used to-date for FRP SIP bridge deck
applications. In some cases, the production tools can be reconfigured for different plate thicknesses and depths, while in others, the geometry is fixed by the manufacturer.

The T-up system (Figure 2-1a) exists in a number of depths and thicknesses from lightweight members to substantial sections designed for all FRP bridge decks (Nelson and Fam, 2012). Due to its geometry, this system promotes good mechanical interlock between the SIP form and concrete, reducing the need for bonding agents. The panels are composed of layers of unidirectional E-glass fibers in alternating longitudinal/perpendicular orientation in addition to layers with randomly oriented E-glass rovings. These systems are pultruded with either polyester or vinylester resin.

The box stiffened plate system (Figure 2-1b) is composed of a solid plate reinforced by hollow box sections spaced 9 in. (229 mm) apart. It is produced using the pultrusion process and has seen the most extensive application to date (Reising et al., 2004; Deiter et al., 2002), partly because the low overall depth of the FRP panel allows additional reinforcement to be placed low in the slab. The system is commercially known as the Composite Deck Solutions (CDS) system. The product is composed of roving and multi-directional E-glass fabric pultruded with a blended polyester-vinylester resin (Dutta et al., 2002).

The system shown in Figure 2-1(c) is fabricated by adhesively bonding a thin pultruded FRP plate to the bottom of prefabricated FRP reinforcement grid and can be considered a modification of traditional grid reinforcing systems (Matta et al., 2006). These grids, composed of glass fibers and vinyl ester resin, are off the shelf decking/grating systems. They are marketed under the trade name GRIDFORM™ by Strongwell. They can be designed and analyzed essentially as FRP grid reinforcement, with the plate providing the function of formwork.

Work at the University of California San Diego led to the development of the system shown in Figure 2-1(d). It was developed to be compatible with custom FRP girders for research on modular bridge systems (Cheng et al., 2005). The plate is composed of unidirectional carbon
fiber and E-glass continuous strand mat. The thin ribs are formed of non-structural foam cores bonded to the plate with uniaxially reinforced carbon FRP hats. The components of this system are assembled using the wet layup method.

The dual cavity system (Figure 2-1e) seeks to maximize the efficiency of the system by locating voids in the bottom of the section (Nam et al., 2006), similar to the Hillman Murray system (Hillman et al., 1990). The FRP components consist of glass fibers and polyester resin and are fabricated by the pultrusion process.

Corrugated plate systems (Fam and Nelson, 2012) (Figure 2-1f) seek to replicate the success of composite concrete decks already used extensively in building construction, and to a lesser extent, bridge decks (Deiter et al., 2002; Hanus, 2006). They benefit from the availability of off-the-shelf corrugated sheets with various profiles. The product (Fam and Nelson, 2012) is composed of alternating unidirectional and random mat E-glass layers. The product has been manufactured with either polyester or vinylester resin.

Prefabricated composite FRP systems exist which seek to optimize the performance of the section by locating concrete and FRP in an ideal configuration (Kitane et al., 2004; Deskovic et al., 1995). However, these systems are essentially precast/prefabricated products as opposed to FRP SIP form systems and are not discussed in this thesis.

Early conceptual work on FRP SIP forms for bridge decks was reported in 1989, along with other FRP applications and a brief optimisation study (Bakeri, 1989). The work was mainly theoretical and a number of different applications were suggested but not pursued. However, it was recognized that bridges presented a suitable application for the technology.

Around the same time, a system was suggested (Hillman and Murray, 1990) with more practical considerations in mind. It consisted of a pultruded plate section, stiffened with T ribs on both the top and bottom. The plate was proposed as SIP formwork for shallow concrete decks for spans up to 83 in. (2.1 m). The system would reduce dead weight by 50-60% but was at the time
generally ill-suited for building applications due to a number of structural performance issues which have since been investigated thoroughly.

A Japanese system developed in the 1990s included an FRP plate on the soffit of an RC slab (Ishizaki et al., 1994). The system consisted of a pultruded plate stiffened with unidirectional I-beams in a manner similar to the system shown in Figure 2-1(c). In this system, the FRP components were not considered to contribute structurally to the deck.

Breakthrough work in the area of FRP SIP forms for concrete decks came in 1998 as a result of an off-the-shelf product becoming available (Hall and Mottram, 1998). The product in this case is a walkway plank composed of a plate stiffened by unidirectionally aligned T-shaped ribs spaced every 2 in. (50 mm). One-way slabs were formed in multiple configurations with concrete being cast on top in various depths (Figure 2-2). The study determined many basic properties of the system and showed that conventional assumptions used in RC design were still valid in such structures.

2.4 Experimental testing and validation
SIP structural forms require some unique detailing when being applied to slab on girder bridge construction. The detailing is depending on the type of girder.

2.4.1 Decks on steel girders
In 2006, a study was conducted which examined the performance of an FRP SIP form system with respect to requirements established by a state department of transportation (DOT) in the USA (Alagusundaramoorthy et al., 2006). A group working at the University of Kentucky partnered with the Ohio DOT to demonstrate the structural performance of a number of FRP deck systems. In this case, 16 full scale one- and two-span specimens (with decks spanning 2 and 3 girders respectively) were tested using specified performance criteria. The SIP form panels were pultruded tube-on-plate design, with the plates acting as structural forms, replacing the bottom reinforcing mat. Top reinforcement for crack control and negative moment was included in the
form of GFRP rebar. These composite decks were compared to both conventional RC bridge decks and Ohio DOT criteria for flexure, shear and deflection. The decks easily met the requirements for shear and flexure, exhibiting safety factors of at least 3. Maximum measured strain in the FRP panels under the flexural loading case was limited to 6% of ultimate under [Live+Impact+Dead] loads and just 1.2% of ultimate under dead load. The required target load was calculated as 1.3[1.67(Live+Impact+Dead)].

2.4.2 Decks on precast/prestressed concrete I-girders
Implementation of FRP SIP forms on precast concrete girders has received recent attention, highlighted by a field project in Wisconsin (Oliva et al., 2007). Two separate FRP products were considered in laboratory testing (Ringelstetter et al., 2006). The first was the CDS system (Figure 2-1(b) and Figure 2-3(c)), and used a pultruded GFRP plate unidirectionally stiffened with 3 in. square tubular box sections, spaced at 9 in. (230 mm). The 18 in. (460 mm) wide panels were formed with a ship lap joint at each edge to ensure a proper seal and avoid grout leakage at the panel to panel connection. The second system consisted of an off-the-shelf FRP decking product, 2 in. (50 mm) deep, comprised of a thin plate unidirectionally stiffened by T-shaped ribs, spaced at 4 in. (100 mm) (Figure 2-1(a)). In both systems, the decking plate provided the transverse bottom reinforcement, while a prefabricated FRP grid was placed for top reinforcement. A third system constructed with GRIDFORM™ FRP grids (Ringelstetter et al., 2006), (Figure 2-1(c)), was investigated whereby a thin GFRP plate was bonded to the bottom of a layer of the grid reinforcement used above, and this new section was used as SIP reinforcement.

The deck systems were tested at full scale on single span arrangements, with an 8 ft. (2.44 m) simple span being supported at their ends along the width of the specimen. All three systems exhibited satisfactory strength, well in excess of the factored loads, with at least 4 times service load supported by each system. Figure 2-4 (curve ‘a’) shows the load-deflection response of the CDS system. Deflections were not a concern with maximum service load deflections being
no more than Span/3000. Strains in the FRP deck plates were measured and did not exceed 700 micro-strain at service, indicating a substantial reserve capacity.

Recent work at Queen’s University, Canada (Nelson and Fam, 2012; Fam and Nelson, 2012) has explored two novel profiles for structural SIP FRP forms. The first system consisted of a FRP pultruded plate stiffened by T ribs approximately 4.5 in. (115 mm) in depth (Figure 2-1(a) and Figure 2-3b), was found to perform well in excess of Canadian (CAN/CSA-S6-06, 2006) and US (AASHTO, 2007) code requirements for strength and serviceability (Nelson and Fam, 2012). Top reinforcement, proportioned in accordance with Canadian code (CAN/CSA-S6-06, 2006), consisted of a layer of GFRP rebar. Careful detailing of the panel to panel connections allowed them to act as both transverse and longitudinal bottom reinforcement. Using a full scale test setup replicating in-place boundary conditions on AASHTO Type III girders and a 6 ft. (1.83 m) girder spacing, the failure mode was observed as punching shear in all cases. Joint detailing including adhesive and mechanical fastening allowed the system to display significant post-peak deformability. A sample load-deflection response of this system is shown in Figure 2-4 (curve ‘b’).

The second system tested at Queen’s University, had similar support conditions (Fam and Nelson, 2012). The FRP SIP structural system comprised off-the-shelf GFRP corrugated sheet pile sections (Figure 2-1(f)). This system used a very thin (0.17 in. (4.2 mm) thick) SIP form system as primary reinforcement for the slab, which showed that satisfactory performance could be achieved with a much lower reinforcement ratio than is common in structural FRP SIP forms. The sheet piles were corrugated with a 4 in. (100 mm) depth, providing adequate stiffness during casting (construction deflections no exceeding Span/240). Panel to panel connection was accomplished using integral pin-and-eye joints in the sheet piles, eliminating the application of adhesives or mechanical fasteners at panel joints. The system exceeded performance requirements.
for strength by at least a factor of 3 (AASHTO HS25 load plus impact), as shown in Figure 2-4 (curve ‘c’).

**2.4.3 Decks on precast concrete bulb tee girders**

Precast bulb tee girders with very wide flanges (up to 6 ft. (1.83 m) wide) are now seeing use in highway bridges (Oliva et al., 2007). These girders may be spaced closely together resulting in a relatively short clear span between the adjacent flanges. It is proposed that a new approach be developed for designing bridge decks for these short spans (Oliva et al., 2007), including the use of shallow FRP panels as structural SIP formwork. Previous work in Canada (Mufti et al., 1993; Mufti et al., 1998) has demonstrated the potential for an entirely unreinforced concrete bridge deck using arch and tie principles; however, a drawback of this system is the development of a single large crack forming at the center of the span. FRP panels as SIP forms were proposed (Oliva et al., 2007) to provide crack control and achieve construction efficiency. Testing was conducted on one-way deck slabs with spans of 3.6 and 6 ft. (1.1 and 1.83 m). The system was also applied in the Black River Falls Bridge in Wisconsin (Oliva et al., 2007). The only reinforcement present in the slab specimens was the structural SIP form system. Sand and fine gravel were adhered to the FRP surface before casting to promote mechanical interlock between concrete and FRP.

**2.4.4 Other general decking systems**

Several systems have been proposed or implemented whereby an FRP SIP composite deck is constructed on structural FRP members. In one case, tubular FRP box girders are used to support composite deck plates which act as SIP structural forms for a concrete bridge deck (Cheng et al., 2005) (Figure 2-1(d)). In another case, the decking system was examined in isolation from the FRP modular system (Cheng and Karbhari, 2006). Figure 2-5 shows a general schematic of the deck system.
Several additional systems have been developed without consideration of bridge type or nature of supporting girders. One example is the dual cavity system (Nam et al., 2006) (Figure 2-1(e)) currently under development at Hongik University, South Korea. This system seeks to make use of a voided panel structure to put concrete only in areas of compression. Investigation of the system is ongoing. The flexural performance of a sheet pile system (Figure 2-1(f)) was investigated at Tongji University, China (Liu et al., 2010). The study reached several conclusions on FRP surface treatments and their impact on performance. An additional example involves the use of a layer of lightweight concrete (<62 lb/ft$^3$, <1000kg/m$^3$) in contact with the FRP SIP form, followed by a top layer of normal density concrete (Figure 2-6) (Keller et al., 2007). Test results have shown that this concept is limited by the shear resistance of the deck at ultimate and would be feasible for spans below 10 ft. (3.05m). The system investigated was highly sensitive to the composition of the lightweight concrete.

### 2.5 Fatigue and cyclic load performance

Recent work has been directed at examining the fatigue performance of FRP SIP composite decks at room temperature. Fatigue testing on a 3 ft. (910 mm) wide, two-span continuous deck was conducted under one-way loading conditions (Dieter, 2002). Testing was conducted on the box stiffened system (Figure 2-1b), cast compositely with a concrete deck. One span was loaded statically throughout the test at 16 kip (71 kN, service load). The second span was subjected to cyclic loading between 4.7 and 21 kip (21 and 93.5 kN) for 2 million cycles. Post fatigue failure testing determined a safety factor of at least 5.43 over service load. Negligible stiffness degradation, even after 2 million cycles, was observed. No cracking of the concrete was observed as a result of cyclic loading.

In a separate study, a full scale two-span deck constructed with the system shown in Figure 2-1d was tested to 2.36 million load cycles using the AASHTO HS20 truck load with impact (Cheng and Karbhari, 2006). The specimen included a bundled carbon-epoxy mesh as
reinforcement in the negative moment zone. Composite action with concrete was achieved by a combination of bonded aggregate and mechanical ribs adhered to the panel. Cracking was found to be limited during 2 million service load cycles to a single hairline crack in the negative moment zone with a width less than the limit prescribed by AASHTO. Displacement response of the system was less than the AASHTO prescribed maximum deflection after 2.1 million cycles and exhibited an increase of 25% over the deflection during initial quasi-static testing. Residual displacements in the system were negligible under all loading regimes, except in the final failure load test. The authors found that the system (Cheng et al., 2005) as a whole performed within existing specifications for conventional RC bridge decks.

Fatigue performance of FRP SIP decks has been examined under severe environmental conditions. A two-span continuous hybrid FRP-concrete bridge deck was constructed (Kwon et al., 2003) on steel girders and tested to 10 million cycles under extreme temperatures (-22°F to +122°F or -30°C to +50°C). This deck was evaluated against a control conventional RC deck tested under identical conditions. Each deck was subjected to a total of 10 million cycles, half at low temperature and half at high temperatures. Loads were applied based on the AASHTO HS20-44 truck with impact at 25.9kip (115 kN). Load-strain responses at low and high temperatures showed that differences in stiffness due to temperature change itself were greater than the cumulative effect of 10 million cycles. Overall conclusions of the study maintain that no unsatisfactory performance was observed in the hybrid deck at extreme environmental conditions even after millions of cycles.

2.6 Analytical modeling
Modeling was conducted by Bakeri (1989) with regard to the original concept for FRP SIP form structures; however, this work focused on the optimisation of variable laminate thickness in a number of inverted vault-like structures. Later, analytical work with more conventional pultruded FRP sections focused on system performance was done by Hall and Mottram (1998). A computer
code was written to take a layer-by-layer incremental approach to section analysis in order to model experimental work underway at the time. Key results of the modeling include capturing of the non-linear load-deflection response of the system and successfully predicting the failure mode. The technique developed is limited to one way bending. It also makes several assumptions on the simplicity of the end restraints which are not reflective of the boundary conditions found in installed bridge decks.

Recognizing that failure in restrained bridge deck slabs occurs primarily in a punching shear mode, a numerical model was developed by Mufti and Newhook (1998) for determining the peak load of a restrained deck slab. The model is applicable to any restrained concrete deck slab with longitudinal and transverse reinforcement subjected to punching shear.

In conjunction with the full scale testing by Cheng et al. (2005) mentioned earlier, the investigators constructed a Finite Element (FE) model, using ply-by-ply failure criteria for the ultimate failure load of the girder in composite action with the FRP SIP form bridge deck.

Numerical analysis of short span FRP SIP form specimens was carried out by Oliva et al. (2007) in conjunction with laboratory testing. Here, simple boundary conditions allowed for the development of a quarter segment FE model. Bond-slip modeling was included between concrete and FRP reinforcement and the concrete material was simulated by a damage plasticity model where two failure modes of tensile cracking and compressive crushing exist. Results of the numerical model compare favourably with experimental data and the authors suggest (Oliva et al., 2007) that this is a satisfactory approach to modeling FRP reinforced concrete bridge decks.

In 2006, Cheng and Karbhari provided a detailed design, analysis and characterization of the FRP girder deck system discussed earlier. A FE model was first developed using a concrete/FRP interface characterization and calibrated based on a series of experimental tests. A parametric study was then undertaken using the FE model to evaluate the effect of panel reinforcement ratio, span to depth ratio and rib spacing. The FE model was implemented in
ABAQUS 2003 and modeled a half span deck. Results of the FE model were found to correlate well with experimental data and suggest that FE models can be used to satisfactorily predict performance given a correct interface characterization.

2.7 Detailing
The installation of FRP SIP form systems is different from the construction of a conventional RC deck. In particular, the use of discrete panels as opposed to orthogonal reinforcement mats presents detailing requirements at girder to panel and panel to panel connections. Details have been developed regarding certain key aspects of FRP SIP forms. It is useful to consider some of these details, particularly with respect to girder-panel connection and panel-panel connection in the context of the superstructure type of the bridge.

2.7.1 Decks on steel girders
FRP SIP form systems do not prevent the use of crown or cross slope details in a concrete bridge deck. For a straight roadway alignment, these systems are just as versatile as conventional RC decks. FRP SIP systems are more often installed using the FRP form plates to bridge between individual girders, not continuously across the entire structure as is done with all-FRP bridge deck systems (Alagusundaramoorthy et al., 2006). Figure 2-7 illustrates several common details for steel and concrete superstructures.

Designers for the Salem Bridge project (Reising et al., 2004) welded leveling angle plates to the top flanges of the girders. These acted as haunches on which the form plates sat with a portion of the plate overhanging the haunch. The space between the leveling plates was then grouted for bearing, and concreting proceeded as normal. Construction considerations will often determine whether discrete plates should be used for each span (Figure 2-7a), or whether continuous FRP SIP form can be used over the girders with drilled holes to accommodate shear studs (Figure 2-7b).
2.7.2 Decks on concrete girders

The installation of FRP SIP form decks on precast concrete girders is similar in concept to steel. Styrofoam haunches are used along the outer edge of the concrete girders (Deiter et al., 2002). These haunches are cut to variable height to accommodate cross slope in the final bridge deck. FRP panels are then set on these haunches, with some length of plate extending past the foam haunch. The gap beneath and between panels is then grouted to provide an adequate bearing surface. Alternatively, this gap can be filled during the concrete pour itself. The panels are not fastened to the girders in any other way. This technique was applied in the installation of the Wisconsin US-151 Bridge (Deiter et al., 2002). In the case of corrugated FRP panels used as SIP forms at Queen’s University (Fam and Nelson, 2012) additional trapezoidal foam dams were cut and fit under the crest of the corrugation to prevent concrete from escaping.

2.7.3 Fascia girders

Fascia girders require unique detailing from interior girders. The normal practice of using SIP forms as shown in Figure 2-7(a) will not work in this case. Two systems have been used to solve this problem. In some cases, longer FRP SIP forms have been used continuously over the facia girder, to support the concrete in the cantilever span. This allows the use of SIP forms for the entire bridge deck forming, but it is necessary to drill holes in the form panels to allow for grouting above the facia girder span (Figure 2-7(b)). Alternatively, the SIP forms can be used for interior spans only, with conventional formwork being used for the cantilever span.

2.7.4 Establishing composite action

Composite action between FRP form and the concrete component of the deck is critical where the form panels are designed as reinforcement. Honickman et al. (2009) isolated this parameter with a series of flat slab tests. The slabs, reinforced only by flat GFRP plates, were tested in shear and flexure. Four bond mechanisms were investigated including GFRP and steel mechanical studs, wet adhesives and bonded aggregates. The bonded aggregate system consisted of 0.16 to 0.35 in.
(4 to 9 mm) silica aggregate bonded to the form panels by SikaDur® 30 epoxy. In the study, both wet adhesive bonding and bonded aggregates eliminated all shear slip before direct shear failure. At the point of failure, delamination occurred within a layer of mortar immediately adjacent to the bond line. Deiter et al. (2002) conducted fatigue tests to 2 million cycles on the CDS system (Figure 2-1b). The cycles were conducted at the AASHTO HS20-44 service load level. Bonded aggregates were used to promote composite action between the concrete and the FRP panels. The aggregate surface consisted of 1/4 in. (6.35 mm) aggregate bonded to the form using Concresive® 1090 epoxy. The testing showed no evidence of bond degradation or departure from composite action. The bonded aggregate method was examined under freeze-thaw conditions in a separate study (Helmueller et al., 2002). After environmental conditioning, interfacial bond stress in a series of push-through shear tests was determined to be at least twice that of the stress encountered during full scale testing of bridge deck, demonstrating the resilience of the system.

Composite action between bridge girders and SIP form deck systems can be achieved. The FRP deck panels are not continuous over the girder tops, allowing full shear connection between the deck and the girders. In an assessment of an already installed bridge using the CDS system (Figure 2-1b), Foley et al. (2010) determined that the bridge’s precast concrete girders were fully composite with the cast in place FRP SIP form deck.

2.8 Design of FRP SIP form deck systems

Typically, field applications of FRP SIP form decks have been largely based on results from laboratory tests in relation to some specifications released by the local state DOT. This is due to the fact that no design aids exist, specifically dealing with concrete structures reinforced by pultruded plate configurations. Provisions do exist for FRP reinforced structures, primarily produced by ACI Committee 440 in North America.

2.8.1 Ultimate strength – design for punching shear
Punching shear is found to govern in many bridge deck applications, especially as the girder to girder span is reduced. The ACI 440.1 R-06 (ACI Committee 440, 2006) design guide equations for shear capacity are based on the elastic neutral axis depth of the cracked section calculated at the cracking moment, and an empirically derived constant relating the section’s concrete strength, shear plane width and neutral axis depth (Hanus et al., 2008). The basic ACI 440 equation for shear in concrete slabs is:

\[ V_c = 10 \sqrt{f_c' b_o c} \quad \text{(U.S. Units throughout)} \quad (1) \]

and is modified for axial stiffness of FRP reinforcement as

\[ V_c = \left( \frac{5}{2} k \right) 4 \sqrt{f_c' b_o c} \quad (2) \]

where \( k \) is the ratio of elastic neutral axis depth to effective reinforcement depth calculated above, \( f_c' \) is the concrete compressive strength and \( c \) is the depth of the neutral axis. \( b_o \) represents the perimeter of the shear surface, calculated at \( d/2 \) from the edge of the loading surface, where \( d \) represents the effective depth of the section. This method, in conjunction with an allowance for axial loads, was shown to be a conservative prediction of experimental results by Hanus et al. (2008).

Deiter (2002) suggested the use of the basic ACI 318 punching shear formula as follows:

\[ V_c = 4 \sqrt{f_c' b_o d} \quad (3) \]

and found this method to be unconservative for some SIP reinforcement types as the shear crack would tend to propagate along the concrete/reinforcement interface. In cases where no bond or only partial bond is provided between FRP forms and concrete, the authors suggest that a coefficient of 0.6 is applied to the calculated \( V_c \) to ensure a conservative prediction.
Cheng and Karbhari (2006) went a step further and developed a simplified design approach based on existing equations and a regression against experimental data and a previous parametric study. The data set used was established using a FE model.

Most recently, a thorough analysis (Bae et al., 2011) was conducted of a system of FRP SIP forms for bulb tee girders. The analysis uses FE and proposes a strut and tie model and a simplified capacity equation.

### 2.8.2 Ultimate strength – design for flexure

In order to apply the principles of the ACI 440.1 R-06 (ACI Committee 440, 2006) design guide, the location of the neutral axis of the member at the limit state must be known (Hanus et al., 2008). This is accomplished by using standard methods available for FRP reinforced concrete members in flexure and accumulating the FRP plate profile into an effective area of reinforcement.

Using the neutral axis depth and material properties for the concrete and FRP used in the system, the moment capacity under pure positive bending can be calculated as follows:

\[
M = A_{frp} E_{frp} \varepsilon_{cu} \left( \frac{d_{frp} - c}{c} \right) \left( d_{frp} - \frac{\beta_1 c}{2} \right)
\]

(4)

where \( \varepsilon_{cu} \) is the ultimate strain of concrete, and the elastic modulus of the FRP used \( (E_{frp}) \) must be known. For additional layers of reinforcement, the moment capacity contribution of each layer can be summed using the above equation to assess the total moment capacity. This method was used to predict moment capacity for a number of laboratory specimens (Hanus et al., 2008), and showed a very good correlation. This method applies to simply supported boundary conditions and decks in most bridges are likely to be continuous over the girders, making this method highly conservative and less accurate. In such continuous deck cases, a more detailed method considering the negative moment contribution over the girders could be undertaken.
2.8.3 Serviceability – crack control
Crack control for the top of the deck slab can be achieved using conventional reinforcement, regardless of the layout of the SIP form system. This reinforcement is also required for resisting negative moments over girders. It would be in the designer’s interest to select GFRP rebar for this purpose, so as not to compromise the durability of the structure. Work by Oliva et al. (2007) demonstrated that SIP formwork itself provides effective crack control for deck soffits provided that there is reasonable bond between the form panels and the concrete deck. In their study, the authors used bonded aggregates for this purpose.

2.8.4 Serviceability – impact factor
Data from field bridge monitoring was used by Reising et al. (2004) to calculate impact factors for the bridge deck. Various methods for calculation were discussed and in this case, the impact factor was strain based and taken as:

\[
IM = \left( \frac{\varepsilon_{\text{dyn}} - \varepsilon_{\text{stat}}}{\varepsilon_{\text{stat}}} \right)
\]

(5)

where \( \varepsilon_{\text{dyn}} \) is the maximum strain during dynamic load testing (35 mph (56 km/h) truck passes) and \( \varepsilon_{\text{stat}} \) is the maximum strain due to static loading (identical vehicle, stationary). Results from this procedure showed an average impact factor of 0.17 for the FRP SIP form deck, well below the AASHTO LRFD limit of 0.33 and also below the AASHTO calculated impact factor of 0.2 for a conventional RC deck of identical layout.

2.8.5 Serviceability – distribution factor
Strains from field load testing have also been used to compute girder distribution factors (Reising, 2002). The factor was based on strains measured on the bottom flange of the six steel girders supporting the deck, where:

\[
DF_{ij} = \frac{\sum_{k=1}^{6} w_k (\varepsilon_k)(N)}{\sum_{k=1}^{6} w_k (N)}
\]

(6)
\( \varepsilon_i \) is the measured girder strain; \( w_i \) is the ratio of inertia of the current girder to that of a typical interior girder; the denominator is the sum of the girder strains for the given truck load and \( N \) is the number of simultaneously loaded lanes (three in this case). Results of this process suggest that the distribution factors meet AASHTO limits of 0.79 for the layout under consideration.

### 2.9 Field installations

#### 2.9.1 Salem Avenue Bridge, Ohio

A field application was undertaken by the Ohio DOT in which the deck of a 683 ft. (208 m) long, 48 ft. (14.6 m) wide, five-span bridge was completely replaced. The replacement, in Dayton, Ohio, used the box stiffened FRP SIP form system (Reising et al., 2004). Numerous construction details pertinent to FRP SIP composite deck installations were validated in this application by monitoring structural responses in the bridge structure after installation. The system was installed in a similar manner to conventional RC construction with GFRP structural plates taking the place of formwork, as in Figure 2-1(b). Study concluded that the FRP SIP composite deck performed the best out of four alternatives implemented in terms of thermal issues, impact factor and panel-to-panel and panel-to-girder connections. The deck was also found not to have a significant impact on the girder distribution factors and otherwise mimicked the performance of the conventional RC deck.

#### 2.9.2 Route US-151 Bridge, Wisconsin

FRP SIP forms were used in the construction of the new route US-151 Bridge B-20-133 North of Waupun, Wisconsin (Berg et al., 2005). The bridge was constructed in 2003 and consisted of two continuous spans measuring 108 ft. (33 m) with a 42 ft. (12.8 m) wide deck. Precast concrete girders were spaced at 2650 mm center to center and a 200 mm thick concrete deck was installed. The deck employed the box stiffened plate system (Figure 2-1(b)) to form SIP formwork. A
combination of FRP bars and grid was then used as additional deck reinforcement. The bridge was constructed successfully with an overall reduction in labor of 57% (Berg et al., 2005) as compared to a conventional RC bridge deck. Figure 2-8 illustrates the system being installed.

2.9.3 Greene County Bridge, Missouri
FRP SIP forms were selected for a rapid bridge replacement of Bridge 14802301 in Green County, Missouri (Matta et al., 2006). The replacement was conducted in 5 days during November 2005. The FRP system chosen was a two-layer FRP grid providing transverse and longitudinal reinforcement, investigated in conjunction with barriers (Matta et al., 2005). The new deck was 7 in. (178 mm) thick and spanned steel girders at 6 ft. (1.8m) spacing. The bottom grid was bonded directly to 1/8 in. (3.2 mm) thick GFRP plates which formed the SIP form (Figure 2-9). The project realized a construction schedule reduction of 70%, with a comparable reduction in labor, all attributed to the elimination of time consuming formwork setting and removal.

2.9.4 Black River Falls Bridge, Wisconsin
The Black River Falls Bridge was constructed in 2007, employing FRP SIP forms between precast concrete bulb-tee girders (Oliva et al., 2007). The bridge used lightweight stiffened FRP panels similar in geometry to Figure 2-1(a), spanning a relatively short distance between the girder flanges. Tie rods were used between girder webs to restrain the deck laterally, and the FRP form panels were found to participate as crack control reinforcement.

2.10 Inspection and performance of existing applications
A comprehensive assessment was conducted on the Route US-151 Bridge in Wisconsin between 2005 and 2009 (Foley et al., 2012). The assessment included in-situ monitoring and load testing along with a numerical model of in-service stresses. A benchmark condition evaluation of the bridge revealed that it was in very good condition at the time of inspection. Cracking was found to be extensive in the negative moment region although this was also observed in an identical
conventional steel reinforced concrete sister bridge. No degradation of the panels was discovered in the inspection. In situ load testing of the bridge indicated that there had been no observable degradation in the load transfer mechanisms within the bridge superstructure. Research completed as part of the program showed degradation of mean shear strength between panel and concrete of 16% after exposure to moisture and 100 freeze thaw cycles. The computer modeling conducted led the authors to conclude that this level of degradation was not of sufficient magnitude to raise any concern over long term performance.

2.11 Cost Effectiveness
An economic evaluation was carried out for the construction of the US-151 bridge presented above (Deiter et al., 2002). The total material cost for the FRP bridge totalled $633,000, while the conventional concrete twin bridge cost $392,000 (total materials), representing a 61% cost increase. The authors attributed this higher cost to the limited availability of the FRP material and the lack of competitive bidding between FRP manufacturers. It is suggested that increased material costs could be offset by decreased labor cost, as the FRP SIP form (the CDS bridge deck system, Figure 2-1(b)) required only 310 man hours to install compared to 713 for the conventional twin bridge deck. Finally, concrete placement was 75% faster with the CDS system.

Cost assessment of the Green County bridge project (Matta et al., 2006; Ringelstetter et al., 2006) showed a total deck cost of $38/ft² of which the reinforcing materials represent a large portion. This represents a roughly 24% decrease in deck cost from the CDS system above. The GRIDFORM system deck took 70% less time to install than a conventional concrete deck and reduced labor costs by 75%. Depending on the cost of labor, this FRP SIP form system can be competitive with conventional reinforced concrete. The authors note that the inherent durability of FRPs increases the cost competitiveness of the technology although it is difficult to quantify any such gains at the time of construction.
2.12 Summary

The state-of-the-art of Fiber Reinforced Polymer (FRP) composite Stay-in-Place (SIP) structural form systems for bridge decks have been presented in this article. The advantages and limitations of the technology were presented, along with the current progress of experimental and analytical investigations. A variety of system configurations were discussed.

Conclusions from research efforts have been very promising and a number of field applications have successfully demonstrated the feasibility of the system. There are nevertheless several areas requiring future work in order to achieve more widespread application of the system. These areas are:

1. Efforts need to be made to synthesize current design methods in order to produce concise design guidelines for FRP SIP structural form systems. Such a guide should encompass all the commercially available systems. The development of simplified analysis procedures to efficiently design SIP systems without the need to resort to detailed FE analyses is recommended.

2. Additional assessments of the in-place bridges built using the FRP SIP systems should be carried out to establish the actual durability of the system. Some field projects have been in place for at least 10 years and such an assessment would be an excellent indication of the system’s longevity.

3. Work should still be conducted on the structural response of bridges built using FRP SIP structural form technology, at ultimate limit states. Thus far, global structural response has only been assessed at service loading in field applications.

4. A comprehensive economic life-cycle cost analysis of the technology with reference to field projects could improve the case for contractors and owners to specify FRP SIP form systems for infrastructure projects.
5. Concerns have been raised related to the ingress of moisture and possible frost jacking between FRP the form and concrete components of SIP form decks (Foley et al., 2010). While this issue has not been encountered in field applications, it is a problem common to steel SIP formwork systems and similar mitigation tactics could be employed.

6. Fire resistance is a concern for any system using exposed FRPs, but this has not been a barrier to the completion of field projects in the past and is largely at the discretion of the local department of transportation.

2.13 Acknowledgements
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2.14 References


Figure 2-1: a) T-up stiffeners on plate b) Tubular sections on plate c) FRP grid bonded to plate d) Ribbed plate e) Dual cavity system f) Corrugated FRP plate formwork.
Figure 2-2: Hall and Motttram FRP SIP formwork system, adapted from (Ishizaki et al., 1994).

Figure 2-3: FRP SIP structural formwork systems: a) Plate with T-ribs (Strong, 2007) b) Corrugated FRP plates (Fam and Nelson, 2012) and c) Tube stiffened plate with FRP grid reinforcement (Reising et al., 2004).
Figure 2-4: Load deflection curves: a) CDS system (Reising et al., 2004) (8 ft (2.44 m) span) b) T-rib system (Strong, 2007) (6 ft (1.83 m) span) c) Corrugated plate system (Fam and Nelson, 2012).

Figure 2-5: FRP SIP formwork system from Cheng et al. (2012).
Figure 2-6: Lightweight and normal density slab with FRP SIP form (Cheng and Karbhari, 2006).
Figure 2-7: Common detailing used in FRP SIP form applications (Nelson and Fam, 2012).
Figure 2-8: FRP SIP form system during installation (Berg et al., 2006).

1.5” I-bars (4” on-center perpendicular to traffic)  1/8” thick adhesively bonded plate

Three-part 0.6” cross rods (4” on-center parallel to traffic)  Vertical shear connectors

Figure 2-9: FRP grid and SIP form system (Zsutty, 1968).
Chapter 3

**Structural GFRP permanent forms with T-shape ribs for bridge decks supported by precast concrete girders**

3.1 Introduction

Stay-in-place (SIP) formwork has been widely used in the construction of concrete slabs, particularly for buildings and bridge decks. By using integral formwork for slabs, construction time can be reduced and labour savings can be realised. The goal of using SIP structural forms is not only to provide integral formwork for the slab, but also to act as the bottom layer of reinforcement, thereby realising increased benefits. Thin precast prestressed concrete panels have been used as SIP formwork for bridge decks (Kluge and Sawyer, 1975), with design procedures presented in the PCI Bridge Design Manual (PCI Bridge Design Manual, 2001). Approximately two thirds of the US DOTs have adopted corrugated steel SIP forms for bridge decks (Grace et al., 2004), however, these systems present two main disadvantages (Taly, 1998); namely, lack of composite action with the concrete deck and corrosion problems. In order to address these issues, SIP forms have been developed using Fiber Reinforced Polymers (FRPs). FRP SIP forms have been explored for simply supported one-way slabs (Hall and Mottram, 1998; Honickman and Fam, 2009) and two-way slabs (Dieter et al., 2002; Alagusundaramoorthy et al., 2006). Corrugated FRP SIP forms with pin-and-eye connections have also been tested for bridge decks supported by precast concrete girders (Fam and Nelson, 2011). FRP SIP forms have been applied in some limited field projects (Berg et al., 2006; Matta et al., 2006).

‘All-FRP’ light-weight bridge deck systems have also been developed (GangaRao and Siva, 2002), and produced either by the pultrusion process, vacuum assisted resin transfer

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moulding, or hand layup technology. These deck systems have dominantly been constructed in conjunction with steel bridge girders, where multiple spans are bridged by single panels with pre-drilled holes to accommodate shear studs (Reising et al., 2004). This installation process is not suitable for decks on concrete girders because the spacing of protruded stirrups is usually not as uniform as studs in steel girders.

This thesis studies the performance of two FRP bridge deck systems installed on concrete girders using full scale experimental testing. The first system comprises commercially available FRP ribbed panels used as SIP structural formwork for a composite concrete deck. The second is a fully hollow FRP deck system joined monolithically to the supporting concrete girders. This system is currently used in practice, and hence, was tested here as a control/reference system, since it uses the same ribbed panels adopted as SIP form system in the proposed concrete deck. The primary objectives are to examine the effects of adding concrete to the structural performance of the system, and also to develop and test the “all-GFRP” deck connection to precast concrete girders, since all current applications have been on steel girders. In both systems, the FRP panels are installed in a single span configuration, spanning between the supporting girders and joined to each other by lap-splices using adhesive and mechanical fasteners. Continuity in the negative moment regions over the girders is provided by a top layer of GFRP rebar in the first system and by a top GFRP plate in the second. The study evaluates these systems based on code requirements and relative to a comparable conventional reinforced concrete deck that was also tested. Additional auxiliary specimens are tested to better understand the lap-spline behaviour of the FRP panels.
3.2 Experimental program

The following sections provide details of the experimental program, including test specimens, materials, fabrication techniques, connections detailing and experimental procedures.

3.2.1 Test specimens and parameters

The experimental program encompasses five primary full scale bridge deck specimens and four auxiliary specimens (Table 1). The deck specimens represent a section of a bridge deck spanning two adjacent concrete girders, where the direction of traffic is parallel to the girder supports. The specimens can be summarized as follows:

3.2.1.1 Primary specimens

The primary specimens D1 to D5 (Table 1(a)) consisted of full scale sections of bridge deck supported by concrete girders, spaced at 1772 mm, center-to-center, with a clear spacing of 1372 mm, and a 1625 mm width in the direction parallel to traffic. All specimens except D4 were detailed such that they were integrated monolithically with the girders, in order to accurately simulate actual field conditions in slab on girder bridges. All primary specimens were loaded using a single AASHTO 500x250 mm load pad through a 12.7 mm thick layer of neoprene to simulate the wheel load of a design truck. It is conceivable that a tandem set of truck wheels (in the circumstance of a dual axle) could be more critical than a single pad under some circumstances, but the practice of using a single tire pad is supported by all literature on the subject.

Control specimen D1 was a traditional steel-reinforced concrete (RC) deck designed according to the Canadian Highway Bridge Design Code (CAN/CSA-S6-06, 2006), and was fabricated as part of another study (Fam and Nelson, 2011). The final design was a 225 mm thick deck with 0.32% steel reinforcement ratio in each direction (CHBDC Cl. 8.18, i.e. 10M @ 150 mm in each direction), top and bottom, with a 40 mm clear cover to the outermost layer. The deck
reinforcement was designed as per the empirical method available in the Canadian code, although conventional flexural reinforcement would perform similarly for comparison purposes.

Specimens D2 to D4 used commercially available ribbed GFRP sections (Figure 3-1 (a and d)) as a novel structural SIP formwork system (Figure 3-1 (b)) to support construction loads and concrete during pouring, as well as to replace the bottom layer of conventional reinforcement. Specimens D2 to D4 had the same span and width as D1. The overall thickness of decks D2 to D4 was selected so that the effective depth to the bottom GFRP plate would be equal to the effective depth of the bottom layer of steel reinforcement in D1, 186 mm. While specimen D1 was designed according to the CHBDC code, specimens D2 to D4 were built using the commercially available GFRP forms, which led to a higher reinforcement ratio than control specimen D1. Furthermore, the GFRP forms had additional vertical ribs.

Specimen D5 was a hollow ‘all-GFRP’ bridge deck. The ribbed GFRP panels were joined to flat, 13 mm thick, GFRP cover plates using mechanical fasteners, forming a lightweight hollow deck (Figure 3-1 (c)) that is currently commercially available. A polymer concrete wearing surface of 9.5 mm thickness was then applied to the top surface to simulate actual practice. The overall thickness of deck D5 (137 mm) was governed by the depth of the pultruded panel plus the cover plate and then the wearing surface layer. For specimens D2 to D5, a panel-to-panel lap-splice detail consisting of epoxy adhesive and self-tapping mechanical fasteners was used (Figure 3-1 (e and f)).

Specimen D2 represent what is envisioned to be a practical system for field application, where the concrete cast onto the GFRP form is monolithically integrated with the supporting girders through the protruding stirrups of the girders. Also, a top layer of orthogonal 12.7 mm diameter GFRP rebar, spaced at 175 mm, is provided for continuity over the supports in the negative moment region, as well as for temperature and shrinkage crack control. This reinforcement was designed according to section 16 of CHBDC (CAN/CSA-S6-06, 2006), with a
clear concrete cover to the outermost layer of 35 mm. To examine the effect of this top layer of GFRP rebar on stiffness and strength, specimen D3 was identical to D2, except it lacked the top reinforcing cage. To examine the effect of the monolithic connection between the deck and supporting girders on stiffness and strength, specimen D4 was similar to D2, except the GFRP SIP form was simply supported on neoprene bearings without any monolithic connection. This parameter is critical as many bridge deck experiments are conducted using simple supports that are not representative of field conditions.

The specific objectives of this study can be summarized as follows: (1) establish the procedure, detailing and assess feasibility of construction of this new bridge deck system, including connection to precast concrete girder, (2) evaluate the strength, failure mode, and service load deflection of the new deck (specimen D2) in light of code limits of deflection and standard truck loading requirements, (3) assess the contribution of top GFRP rebar towards overall strength, with emphasis on negative moment contribution (specimens D3 vs. D2), (4) assess the impact of monolithic connection to girders on overall strength, with emphasis on the end restraints (specimens D4 vs. D2), (5) evaluate the strength, failure mode and service load deflection of an “all-GFRP” deck commercially available that uses the same panels used in the proposed SIP form system (specimen D5) for the same span, in light of code limits of deflection and standard truck loading requirements.

3.2.1.2 Auxiliary specimens
The auxiliary specimens A1 to A4 (Table 1(b)) were tested to investigate the transverse panel-to-panel connection (Figure 3-1 (e and f)) featured in the primary specimens as this connection was found to govern their ultimate capacity. While the test setup of this connection is simplified as one way-bending, relative to the condition in the deck, it is still valuable in that it helps better understand the failure mechanism of the connection. This scheme is also simple enough to provide a first step validation of analytical strength models governed by this type of connection
failure. The 1625 mm long beam-like specimens were detailed such that the splices were located at mid-span and all tensile forces transferred directly across the splice, thereby inducing a direct joint failure (Figure 3-2). The specimens consisted of narrow (152 mm wide) sections of the large deck specimens, such that the span is parallel to the direction of traffic. The 152 mm width represents the typical spacing of the size 14 (3.2 mm diameter) mechanical fasteners used in the decks. Specimens A1 to A3 represented sections of the concrete-filled SIP form system, while A4 reflected the hollow ‘all-GFRP’ deck. Table 1(b) provides the splicing method used for each specimen. All auxiliary specimens were loaded in a simply supported four-point bending scheme (Figure 3-2).

3.2.2 Materials

Figure 3-3 shows a summary of the stress-strain curves of the GFRP materials and steel rebar used in this study, which are described as follows:

3.2.2.1 GFRP ribbed panels

A standard class of GFRP ribbed panel was used in the fabrication of specimens D2 to D5 and A1 to A4. The panel is 114 mm deep, 838 mm wide and is fabricated using the pultrusion process. The panel has a 12.7 mm thick plate section integrated with “T-up” ribs spaced at 203 mm (Figure 3-1 (a and d)). The ribs are 12.7 mm thick and their flanges are 102 mm wide. Along both edges of the panel, the bottom flange plate thickness is reduced to one half within a 51 mm length to allow for a lap-splice connection between panels (Figure 3-1(f)). The sections are produced in a continuous pull-and-cut to specified length process, using E-glass fibres and polyurethane resin. Burn out tests revealed a [0/40/90/130] fibre orientation in the bottom plate section with woven mats interspersed. Material testing was conducted according to ASTM D-3039 for the bottom plate section (Figure 3-3) and revealed ultimate tensile strength and modulus of 203 MPa and 16.8 GPa, respectively, in the longitudinal direction, and 126 MPa and 15.6 GPa, respectively, in the transverse direction.
3.2.2.2 GFRP Rebar
V-rod number 4 (12.7 mm) GFRP bars were used as top reinforcement in specimens D2 and D4. The manufacturer reported tensile strength and modulus of the bars are 786 MPa and 46.3 GPa, respectively (Figure 3-3).

3.2.2.3 Concrete
The high early strength concrete used in the support girders was designed and tested to at least 45 MPa using ASTM C39. The concrete used in the deck slabs was specified as 35 MPa at seven days and was tested to between 38 and 41 MPa for all specimens at time of testing. Coarse aggregates were of 19 mm maximum diameter crushed limestone.

3.2.2.4 Steel rebar
Standard 400 grade steel rebar, size 10M, was used throughout the girders and in control specimen D1 deck slab. The rebar was tested as per ASTM A370 and showed a yield stress of 435 MPa (Figure 3-3).

3.2.2.5 Polymer concrete overlay
A commercially available polymer concrete overlay product was used to provide a 9.5 mm wearing surface to specimens D5 and A4 for driveability reasons. The overlay consists of silica stone aggregates suspended in a polyester resin. The manufacturer reported tensile strength is 17.5 MPa and the overlay is assumed to be non-structural.

3.2.3 Fabrication of test specimens
3.2.3.1 Primary specimens
A key consideration in specimen fabrication was to represent accurately the details of field support conditions for slab on precast concrete girder bridges. Simulated AASHTO Type III concrete girders were constructed to provide a properly detailed joint between slab and girder. Protruding stirrups were detailed using common US DOT specifications and were spaced at 150
mm along the centerline of the 400 mm wide girder top flange. The top girder surface was roughened during casting to promote bond at the cold joint. At the base of the girders, conduits allowed 29 mm diameter B7 grade threaded rods to be passed between the two support girders to prevent relative rotation and translation. Once the supports were installed, the decks were fabricated on top of them exactly as they would be in the field. Specimen D1 was constructed conventionally using wooden formwork and steel rebar. The ends of the rebar cage were J-hooked to achieve full rebar development as in a continuous slab and the cage was tied directly to the protruding stirrups (Figure 3-4(a)).

Specimens D2 and D3 were constructed using detailing specifically developed for this project. First, a 12x12 mm foam strip (haunch, shown in Figure 3-4(d)) was placed along each girder’s inner edge for the entire length of girder. GFRP formwork panels were pre-cut to be 150 mm longer than the clear girder spacing, to allow for 75 mm bearing lengths over the girders at each end. The panels were then rested on the foam haunches, resulting in a 63 mm overhang past each haunch. This created a 12 mm gap between the GFRP panel soffit and the precast girder top surface, in which grout was later poured to provide a bearing surface (Figure 3-4(d)).

As the panels were placed, each was joined to its adjacent section at the 50 mm wide lap-slice using adhesive and mechanical connection (Figure 3-1(f)). In this study, two adjacent panels were sufficient to produce the full width of a specimen. The panel-to-panel connection area was adhered with a layer of Sikadur 30 high viscosity epoxy. Once the epoxy had cured, size 14 (3.2 mm diameter) self-tapping screws were installed at 152 mm intervals to provide mechanical connection. The fastener spacing used was recommended by manufacturer of the commercially available “all-GFRP” decking product. Once the panels were joined, the gap between panel and girder was filled using non-shrink grout as indicated earlier.

For specimen D2, the GFRP rebar cage was tied and placed; no chairs were necessary as the cage sat directly on the GFRP panel “T-up” flanges. No bottom layers of rebar were used in
specimens D2 to D4. To accommodate the development length of the top layer of GFRP rebar oriented normal to traffic direction, the deck included 200 mm overhangs beyond the outer edge of the flange of support girder (Figure 3-4(b)). Unlike the steel rebar in control specimen D1, the GFRP rebar could not be bent for anchorage. Because specimen D3 did not include top rebar, the deck was cast such that it terminates at the outer edges of support girders as in D1 (Figure 3-4(a)). The concrete was poured using a hopper (Figure 3-4(b)). A separate section is allocated later to address the bond system and composite action between concrete and FRP form.

Specimen D4 was constructed intentionally to achieve a simply supported condition. Instead of casting integrally to the concrete girders, the GFRP panels were placed on 12.7 mm thick and 75 mm wide neoprene pads resting on the inner edges of the support concrete girders, along the full width, on both sides of the span. Concrete was cast into the GFRP panel only so as to prevent monolithic connection with the form. As such the clear span was similar to all other specimens but the center to center span of the deck, between the neoprene pads, was 1437 mm. The length of GFRP sections and the width of bearing area (75 mm) in specimen D4 were similar to D2.

Specimen D5 was constructed using details unique to this study. The panels were placed on the support girders using the process used for D2 and D3 (foam haunches, adhesive and mechanical fasteners at the lap-splices and grouting). The 13 mm thick top plates were then rested directly on top of the “T-up” flanges and fastened using size 14 (3.2 mm diameter) self-tapping screws (Figure 3-4(c)). This formed a composite hollow GFRP deck with an overall thickness of 127 mm. The joint in the top plate was located along the center of the span (i.e. in compression) and the plates were mated using high viscosity epoxy at this location. Over the concrete girders, the deck was filled completely with concrete through holes pre-drilled in the top plate. Concrete was confined to the girder connection area only, using specially cut foam blocks inserted between the GFRP ribs as dams (Figure 3-4(c and d)). In order to simulate continuity of
the top GFRP plate over the supports, four, 19 mm diameter grade B7 rods, were anchored into the simulated AASHTO girders (Figure 3-4(c)) and allowed to protrude through the top plate. The rods were then torqued to yielding in order to provide a slip critical connection between the top plate and the girders. Finally, a 9.5 mm thick polymer concrete wearing surface was applied to the top surface of the GFRP deck to provide traction and to cover the protruding screw heads. The top GFRP plates were commercially sandblasted prior to application of the wearing layer to promote bonding.

3.2.3.2 Composite action

Composite action between FRP formwork and the concrete component of the deck is critical where the form panels are designed as reinforcement. This concept was explored in depth previously by the authors (Honickman et al. (2009b)). A series of slab tests were carried out to explore four bond mechanisms, including the use of mechanical studs, wet adhesive before casting and adhesively bonded coarse aggregates. The wet adhesive was shown to provide the best performance and ease of fabrication. Also, both wet adhesive and bonded aggregates showed no slip prior to failure by sudden delamination. The delamination occurred within a layer of cement mortar immediately adjacent to the bond line, indicating that the shear capacity of the bond interface was superior to that of the concrete itself. Based on that study, it was decided to use wet adhesive bond in the current study. Within 30 minutes prior to casting specimens D2 to D4, a special low viscosity wet adhesive was used to coat the GFRP panels to provide bond at the panel-concrete interface.

3.2.3.3 Auxiliary specimens

Narrow (152 mm) wide sections were cut from the GFRP panels to be used for specimens A1 to A4 (Figure 3-2). The panels of A1 and A4 were adhesively bonded at the lap splice using high viscosity epoxy, and then fastened using a single self-tapping screw. For A2, only adhesive bond was used at the splice and for A3 only a self-tapping screw was used. Formwork was constructed
around the spliced panels and the surface was coated with adhesive, less than 30 minutes prior to casting concrete. For A4, a 152 mm wide and 13 mm thick top GFRP plate was fastened to the top “T-up” flanges forming a hollow deck section. The top plate was also sandblasted and a 9.5 mm layer of polymer concrete was applied as in specimen D5.

3.2.4 Test setup and instrumentation
Primary specimens D1 to D5 were loaded using a 500x250 mm load pad contacting the specimen through a 12.7 mm thick neoprene pad at the center of the specimen, directly above the panel splices (Figure 3-5). Loading was applied at a rate of 1 mm/min using a 2000 kN hydraulic actuator and monitored continuously through a 2.25 MN load cell. For specimens D2 to D5, longitudinal and transverse panel strains were measured on the slab soffit at various locations along the longitudinal and transverse centerlines, particularly in the vicinity of the panel-to-panel lap splice. For D1, steel rebar strains were measured internally. Strains were monitored using 5 mm electric resistance strain gauges. On the top surface of slabs D1 to D4, strains were measured longitudinally and transversely using 100 mm displacement transducers (PI gauges) and electric resistance strain gauges. Deflections were measured at multiple locations to provide deflection profiles, using 100 mm Linear Potentiometers (LPs). LPs were also used to measure any longitudinal top rebar slip in specimen D2 by extending the rebar ends slightly outside the concrete (see Figure 3-3(g)). Measuring FRP rebar slip for embedded bars using this technique is common and has been reported in literature (Won et al. 2008; Saiedi et al. 2011). Relative slip between the GFRP SIP form and support girders in D3 were measured using similar instruments.

The auxiliary specimens were loaded using a simply supported four-point bending scheme (Figure 3-2) with deflection measured at the centerline by an LP and longitudinal strains measured at multiple locations around the splice.

3.3 Experimental results
The following sections discuss in detail test results of auxiliary and primary test specimens.
3.3.1 Auxiliary specimens

Figure 3-6(a) shows the load-deflection behaviour of the auxiliary specimens A1 to A4 while Figure 3-6(b) shows the load-strain behaviour. The location of the strain gauges in Figure 3-6(b) is similar to that of SGT2 in primary specimens D2 to D5, shown later in Figure 3-14(c) (i.e. at mid-length of lap-splice, mid-way between two fasteners). Specimen A1 shows three distinct phases in performance. Initially, it performs as an uncracked section, and after cracking, the specimen enters a phase dominated by progressive peeling at the splice location. At the point of peak load, the adhesive lap splice delaminates completely and the load drops significantly, to about 25 to 30% of peak value. The specimen enters a final phase where the response is dominated by the remaining mechanical fastener. Specimen A2 is similar except that it has no mechanical fastener; hence the strength is lost completely upon adhesive failure. In specimen A3, the behaviour collapses into the third state immediately after cracking as no adhesive is present. The behaviour based on the fastener is characterized by a very low strength sustained over a large range of deflection. The excessive deflection is a result of a significant slip and tilting of the mechanical fastener at the splice.

The load-strain responses indicate that peeling at the splice location results in a strain reversal at the location of strain measurement; this is used to detect splice peeling in the full scale primary specimens. Complete delamination of the splice occurs at -3000 micro-strains. Specimen A1 exhibited a lower peeling load and hence a smaller stiffness after cracking compared to A2. It is possible that some damage was induced in the adhesive bond of specimen A1 as a result of installation of the fastener. Specimen A4 shows a much softer initial response, due to the lack of concrete, until peeling of the joint initiates. After complete delamination, A4 also collapses into a fastener-dominated response at roughly the same load as A1.

3.3.2 Primary concrete specimens D1 to D4
Figure 3-7 presents the load-deflection responses at the centerline of primary bridge deck specimens D1 to D5 directly under the loading pad. Figure 3-8 presents the load-strain responses in the longitudinal (i.e. deck span direction, normal to traffic direction) and transverse direction (i.e. strain in the direction parallel to traffic), centered on the lap-splice seam, 75 mm from the centerline of the span (half way between fasteners). Figure 3-9 shows the longitudinal and transverse strain responses at a variety of other locations along the soffit of deck specimens D2 and D3. Figure 3-10 shows the PI gauge concrete strains on the top surface of decks D1 and D2 near the loading pads and above the edge of supporting girder. The following sections examine the results in further detail.

From Figure 3-7, it is evident that the load-deflection performances of specimens D2 and D3 with GFRP SIP forms are initially quite similar to that of the RC control specimen D1, in terms of stiffness. Specimen D4 shows lower stiffness initially due to the different support boundary conditions. At a load of about 250 kN, negative moment cracking is observed over the girders, as evident by the PI gauge D2TL in Figure 3-10, at a strain of 130 micro-strains. However, its effect on the overall stiffness (Figure 3-7) of decks D1 to D3 appears to be insignificant. It is worth noting that no slip was measured for the top GFRP rebar in specimen D2, suggesting adequate anchorage within the overhangs. Also, in specimen D3, insignificant slip between the GFRP SIP form and support girders was measured (0.24 mm at peak load).

A critical element of the load-strain performance observed in Figure 3-6(b) is the reversal of transverse strains from tension to compression. The authors designed the auxiliary series of tests described above to examine this phenomenon at the panel splice location. Joint peeling occurs as a moment develops due to the eccentric nature of the lap splice and is influenced by two factors, namely, the stresses acting to pull apart the splice, and the rotation at the splice location, which creates an additional moment on the splice. Examining specimen D5, it can be seen that splice peeling occurs at a relatively low load because the stress at the joint location is higher than
for specimens D2 to D4 at comparable loads, due to a considerably lower transverse stiffness. Additionally, the rotation at the joint is significantly higher at any load level than for D2 to D4 (Figure 3-10). Specimen D4 began peeling at a lower level than D2 and D3 because the monolithic end detailing of the latter specimens can be expected to carry some of the joint splitting force.

3.3.2.1 Failure modes

Control RC specimen D1 failed at 694 kN by punching shear. This failure mode entails a 2-way shear cone forming in the concrete slab, beginning roughly at the perimeter of the loaded area and radiating out and down until terminating at the slab soffit. The cone can be visualized as a 1-way beam shear failure, revolved through 360 degrees. When punching shear occurs, load drops suddenly and completely (Figure 3-7). On the other hand, specimens D2 to D4 with GFRP SIP forms experienced a progressive failure process in three stages as follows:

(i) Between the loads of 300 and 400 kN, the panel-to-panel lap-splices began to peel progressively, leading to slight yet distinct reduction in stiffness at that load range (Figure 3-7),

(ii) At loads of 670 to 730 kN, and deflections of 7 to 10 mm - very similar to control specimen D1 due to their similar effective depths - all three decks D2 to D4 suffered a punching shear failure (Figure 3-11(a)), leading to a significant change in their load-deflection responses. Unlike the conventional RC deck, they exhibited varying increments of reserves in capacity and significant pseudo-ductility thereafter (Figure 3-7). Specimens D2 and D4 particularly had additional capacity of 33% and 20%, respectively, after the punching shear,

(iii) After adhesive peeling at the lap-splice (stage i), the mechanical fasteners remained intact for some time. They did however start to tilt and tear through the GFRP panels, locally, as loading continued, especially beyond punching shear. Eventually, they completely cut through the GFRP lap-splice in a bearing failure mode (Figure 3-11(b)). These localized failures occurred
progressively and are reflected clearly by the sudden progressive load drops in Figure 3-7 beyond the peak loads. The mechanical fasteners themselves remained intact.

It is worth noting that the maximum GFRP longitudinal strains in decks D2 to D4 (Figure 3-9(b)) at peak load did not exceed 5700 micro-strains at ultimate, which is less than half the material ultimate tensile capacity. Also, when punching shear failure occurred, the concrete compressive strain was about -4000 and -2000 micro-strains in specimens D1 and D2, respectively (Figure 3-10).

3.3.2.2 Performance under service loads
At a service load of 122.5 kN representing the CHBDC CL-625-ONT plus impact loading, the control RC deck D1 and D2 with FRP SIP form exhibited maximum deflections of 0.42 and 0.51 mm, respectively (Figure 3-7), while the most stringent common limit for bridges is 0.85 mm for this span (L/1600). The AASHTO HS-25 plus impact loading is only slightly lower than the CL-625-ONT, 115.6 kN. The HS-25 load was chosen for its severity to avoid overestimating the performance of the system and also for its similarity to the Canadian CHBDC service load. An impact factor is added to account for dynamic effects which are not simulated by monotonic testing. Also, from Figure 3-7, the adhesive peeling at the lap-splice in D2, which is detected by the transverse strain reversal, occurs at about 350 to 450 kN, loads that are well above the service load levels. It remains important for future work to verify the integrity of this splice under cyclic fatigue loading.

3.3.2.3 Effect of top layer of rebar reinforcement
Specimen D3 replicates D2, except for the lack of the top orthogonal layer of GFRP rebar to examine its contribution to strength and stiffness. Figure 3-7 shows that the performance of D3 up to punching shear failure is quite similar to specimen D2. However, beyond this level, the specimen does not gain further capacity unlike D2. Instead, the load remains somewhat stable under further deflection, and then drops gradually. It can be seen from Figure 3-8 and Figure
3-9(b) that longitudinal strains at mid-span are higher in D3 than D2. This is perhaps attributed to the fact that upon negative moment cracking over the supports in D3, the system lends itself more towards a simply supported span, due to the lack of top reinforcement and hence reduced end restraints. It is concluded that the presence of top layer of rebar resulted in increasing ultimate strength by 17%.

3.3.2.4 Effect of monolithic connection to girders

Specimen D4 was constructed and tested as a simply supported deck on neoprene pads, as opposed to D2 which was cast monolithically with the girders. The goal was to assess this rather simple approach of testing bridge decks, which has been commonly used by researchers in literature. Figure 3-7 shows a clear reduction in initial stiffness prior to punching shear. The mid-span deflection at service load is 1.38 mm, three times larger than D2 deflection under the same load. It also exceeds the L/1600 deflection limit but satisfies the L/800 limit. The flexural behavior before punching shear is therefore influenced by the fixity at the deck-girder connection. Punching shear failure load is not affected by the boundary conditions yet the post-punching behaviour and ultimate load are markedly different. After punching in D4, the load dropped slightly but then started increasing gradually at higher deflections, until it reached a peak value, 13% lower than D2. Transverse deflection profiles in Figure 3-12 indicate that some uplift occurred at the edges of D4. Transverse strains at the splice reached similar values to D2 and D3 at failure of the first fastener. Longitudinal strains at locations 3 and 5 (same locations in D2 and D3 as shown in Figure 3-9) diverge after punching shear with strain at location 3 reaching 5600 micro-strains and at 5 reaching 1900 micro-strains. This phenomenon is related to the boundary effect at the deck-girder interface. After punching shear, large longitudinal cracks formed over the length of the specimen and allowed separation in the transverse direction similar to that of specimen D3 above. It is worth noting that in the field, the deck surrounding the test patch would
act as a diaphragm to prevent such motions. It is concluded that simply supported decks, ignoring the monolithic connection to girders, while significantly easier to fabricate, overestimate the initial stiffness and deflections significantly at service load level. However, they slightly underestimate the ultimate strength, and hence are considered conservative.

3.3.3 ‘All-GFRP’ primary specimen D5

Figure 3-7 shows that ‘all-GFRP’ deck specimen D5 has a significantly lower strength and stiffness relative to all other specimens. This is attributed not only to the lack of concrete fill but also to the 26% smaller thickness relative to other specimens. Figure 3-13(a) shows the load-deflection responses at various points across the width of the specimen, while Figure 3-13(b) shows the deflection distribution in the transverse direction at service load, at an arbitrary 300 kN load and at ultimate load. Figure 3-14(a) and (b) show the load-longitudinal and transverse strain responses, respectively, at various locations, while Figure 3-14(c) shows a picture of the underside of the deck showing the strain gauge locations near the center of the deck, relative to the mechanical fasteners.

3.3.3.1 Failure mode

The transverse strain behaviour at the lapsplice in Figure 3-8 indicates that the adhesive peeling occurs at about 80 to 100 kN, a much lower peeling load than in specimens D2 to D4 (300 to 400 kN). This low peeling load was also observed in the auxiliary specimen and is due to additional bending at the splice imposed by the large transverse relative deflections, reflected in Figure 3-13. In fact, it is worth noting that transverse strains at location SGT4 at the bottom surface (Figure 3-14(b)) are consistently compressive, as a result of the localized transverse bending near the center of the deck. The lack of transverse webs leaves the hollow deck susceptible to large local deflection and reduces the effectiveness of load distribution considerably. Figure 3-14(b) suggests that early peeling is most pronounced under the point of loading (SGT2), and by the time peak load was reached it has propagated to the quarter span positions (SGT1). The peak load was
reached when the first fastener tear through the GFRP at mid span (Figure 3-14(c)), then the load drops and rises again until a second fastener tears through, and the process continues progressively. This process has resulted in a remarkable pseudo-ductile behaviour (Figure 3-7). Eventually, scattered localized fractures of the GFRP plate (Figure 3-14) were seen. At the peak load, when the first fastener tore through the GFRP lap, the maximum longitudinal strain was about 6500 micro-strains, about 50% of the material ultimate strength.

3.3.3.2 Performance under service loads
At service load, specimen D5 exhibited a maximum deflection of 5.94 mm (Figure 3-7). This far exceeds the limit of L/800 (1.7 mm). Also, from Figure 3-8 the adhesive peeling at the lap-splice, which is detected by the transverse strain reversal, occurs at about 80 to 100 kN, loads that are below the service load levels. As such, performance of the ‘all-GFRP’ deck D5 is not considered satisfactory for that particular girder spacing.

3.4 Conclusions
A new concrete bridge deck cast onto Glass Fibre Reinforced Polymer (GFRP) Stay-In-Place (SIP) structural forms is studied. The forms, which replace the bottom layer of conventional rebar, are essentially flat plates with ‘T-up’ ribs. They span between support girders spaced at 1772 mm, and are lap-spliced using adhesive and mechanical fasteners. Special attention was given to detailing of monolithic connections between the deck and girders to simulating continuity. The flange of the supporting concrete girders had a rough surface finish and protruded steel stirrups and mimicked AASHTO type III girders. Five full scale decks were tested to: (a) compare the novel system with a conventional RC deck, (b) examine the effect of eliminating a top layer of orthogonal GFRP rebar, (c) assess a practice commonly used in bridge deck tests of using simple spans resting on neoprene pads, neglecting connection to girders, and (d) examine an ‘all-GFRP’ deck commercially available, using the same GFRP panels but with a top GFRP plate fastened to the ‘T-up’ ribs. Four auxiliary flexural specimens were tested to examine the
lap-splice. The concrete deck with GFRP SIP form showed 33% higher strength than the conventional RC deck. It also achieved a peak load, 7.8 times higher than service load plus impact. Deflection under service load was less than span/1600. Remarkable deformability and pseudo-ductility was achieved at ultimate, beyond punching shear, due to progressive failure of the lap-splice. The simply supported deck failed at a slightly lower strength than the monolithically-cast one but had a much lower stiffness.

3.5 Acknowledgements

The authors would like to acknowledge financial support provided by NSERC Canada and the Canada Research Chairs program. The authors are also grateful for the valuable assistance provided by Dave Tryon and William VanRuyven at Queen’s University.

3.6 References


Table 3-1 Test matrix and results

(a) Primary specimens (full scale bridge decks)

<table>
<thead>
<tr>
<th>Spec ID</th>
<th>Parameters</th>
<th>Support conditions</th>
<th>Depth (mm)</th>
<th>Span (c.c.) (mm)</th>
<th>Peak load (kN)</th>
<th>Service load (kN)</th>
<th>Failure mode</th>
<th>Deflection at service load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>RC control (Fam &amp; Nelson, 2011)</td>
<td>Monolithic</td>
<td>225</td>
<td>1772</td>
<td>694</td>
<td>CHBDC CL-625-ONT + Impact (122.5 kN) &amp; AASHTO HS-25 + Impact (115.6 kN)</td>
<td>Punching shear and abrupt load drop</td>
<td>0.42</td>
</tr>
<tr>
<td>D2</td>
<td>FRP SIP form &amp; top GFRP rebar</td>
<td>Monolithic</td>
<td>186</td>
<td>1772</td>
<td>911</td>
<td></td>
<td>Progressive tearing of splice between panels</td>
<td>0.51</td>
</tr>
<tr>
<td>D3</td>
<td>FRP SIP form without top rebar</td>
<td>Monolithic</td>
<td>186</td>
<td>1772</td>
<td>767</td>
<td></td>
<td>Progressive tearing of splice/splitting of slab</td>
<td>0.75</td>
</tr>
<tr>
<td>D4</td>
<td>FRP SIP form &amp; top GFRP rebar</td>
<td>Simply supported</td>
<td>186</td>
<td>1437</td>
<td>805</td>
<td></td>
<td>Progressive tearing of splice between panels</td>
<td>1.38</td>
</tr>
<tr>
<td>D5</td>
<td>Hollow ‘All-GFRP’</td>
<td>Monolithic</td>
<td>137</td>
<td>1772</td>
<td>434</td>
<td></td>
<td>Progressive tearing of splice / tearing of bottom plates</td>
<td>5.94</td>
</tr>
</tbody>
</table>

(b) Auxiliary specimens (splice test specimens)

<table>
<thead>
<tr>
<th>Spec ID</th>
<th>Parameters</th>
<th>Support conditions</th>
<th>Depth (mm)</th>
<th>Span (mm)</th>
<th>Peak load (kN)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>FRP SIP form, mechanical and adhesive splice</td>
<td>Simply supported</td>
<td>186</td>
<td>1400</td>
<td>22.9</td>
<td>Sudden delamination followed by progressive fastener tear out</td>
</tr>
<tr>
<td>A2</td>
<td>FRP SIP form, adhesive splice only</td>
<td>Simply supported</td>
<td>186</td>
<td>1400</td>
<td>22.0</td>
<td>Sudden delamination and complete collapse</td>
</tr>
<tr>
<td>A3</td>
<td>FRP SIP form, mechanical splice only</td>
<td>Simply supported</td>
<td>186</td>
<td>1400</td>
<td>7.1</td>
<td>Progressive tear out of mechanical fastener</td>
</tr>
<tr>
<td>A4</td>
<td>Hollow GFRP, mechanical and adhesive splice</td>
<td>Simply supported</td>
<td>137</td>
<td>1400</td>
<td>6.7</td>
<td>Sudden delamination followed by progressive fastener tear out</td>
</tr>
</tbody>
</table>
Figure 3-1: (a & d) Typical GFRP single panel section, (b) cross section of deck specimens D2 and D4, (c) cross-section of deck specimen D5, (e & f) panel-to-panel splice detail.

Figure 3-2: Layout of auxiliary specimens: (a) specimens A1 to A3, and (b and c) specimen A4.
Figure 3-3: Materials properties of FRP and steel used in this chapter. Typical test results shown for steel, FRP plate and form panels; manufacturer’s data displayed for V-rod.
Figure 3-4: Details of primary specimens D1 to D5: (a) conventional RC D1, (b) casting concrete on to GFRP forms for D2 to D4, (c) ‘all-GFRP’ deck D5, and (d) slab-girder connections.
Figure 3-5: Test setups for (a) specimen D2 and (b) specimen (D5)

Figure 3-6: (a) Load-deflection and (b) Load-strain responses of auxiliary lap-splice specimens A1 to A4
Figure 3-7: Load-deflection responses of primary bridge deck specimens D1 to D5

Figure 3-8: Load-GFRP form strain responses of primary bridge deck specimens D1 to D5
Figure 3-9: Load-GFRP form strain responses of primary bridge deck specimens D2 and D3 (a & b)
Figure 3-10: Load-concrete strain responses at top slab surface for specimens D1 and D2

Figure 3-11: Failure modes of primary deck specimens (a & b). Visible in (a) is the top of the punching cone. See Figure 5-3 for an image of the top and bottom of a punching cone.
Figure 3-12: Deflection distribution in the transverse direction (parallel to traffic) for decks D2 (a) and D4 (b)

Figure 3-13: (a) Deflections of ‘all-GFRP’ deck specimen D5 at various locations (b) in transverse direction
Figure 3-14: (a & b) Load-strain responses of ‘all-GFRP’ deck specimen D5 at the bottom surface and (c) failure mode
Chapter 4

A new bridge deck cast onto corrugated GFRP stay-in-place structural forms with interlocking connections

4.1 Introduction
Innovative bridge construction and replacement techniques are being viewed with additional interest due to the ailing state of bridge infrastructure in North America. In particular, solutions are being sought to accelerate construction and extend service life of bridges, thereby lowering their life cycle cost. Reinforcing bridge decks with pultruded FRP bars has already become accepted in some jurisdictions as an alternative to steel rebar (Benmokrane et al., 2007). Pultruded FRP sections and plates of various geometries are commercially available, and “All-FRP” bridge decks have indeed been developed and used (Keller, 2001).

In bridge deck construction, the concept of stay-in-place (SIP) structural forms simplifies and accelerates construction to a great extent. It was first developed as thin precast prestressed concrete slabs used to support the cast-in-situ concrete deck (Kluge and Sawyer, 1975). Given the versatility, lightweight and durability characteristics of FRP sections, some configurations have been introduced as SIP forms for one-way simply supported concrete slabs and girders (Hall and Mottram, 1998, and Honickman and Fam, 2009). Two-way SIP form-concrete slab performance was also evaluated using simply supported boundary conditions with additional FRP grid reinforcement (Dieter et al., 2002). Fatigue performance of the SIP form-concrete system was examined and found to be satisfactory (Cheng and Karbhari, 2006). Two-span arrangements have been examined to assess continuity issues (Alagusundaramoorthy et al., 2006). The system has

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also been examined in a military application as part of a modular all-FRP bridge concept by Hanus et al. (2009). More recently, the system has been used in field applications for concrete decks on steel girders (Reising et al., 2004) and concrete girders (Berg et al., 2006), demonstrating the practicality of the system. A cost analysis by Berg et al. concluded that for the full bridge decking undertaken in their study, construction labour costs were reduced by 57% while material costs increased, concluding that the system had potential to be cost competitive with conventional technology. No work, however, addressed FRP corrugated SIP form systems; likewise, FRP SIP forms for bridge deck overhangs have only been investigated in the context of steel girders (Matta et al., 2005, Matta et al., 2006). Also, very little experimental testing has been conducted using boundary conditions which simulate actual field conditions, including connections to girders.

This chapter describes the application of a new system where corrugated GFRP pultruded plates are used as SIP structural formwork for a concrete deck supported by precast concrete girders. In this configuration, the SIP formwork also acts as the bottom layer of structural reinforcement (SIP structural formwork) for the concrete slab, further optimising the construction sequence. Plate-to-plate connections are made through pin-and-eye joints to insure continuity in the direction parallel to traffic. The system is evaluated experimentally using test setups designed specifically to mimic real applications. Single span FRP-concrete decks were cast integrally to simulate the connection with AASHTO type III precast concrete girders. A cantilevered specimen was also tested. The flanges of the support girders included protruding stirrups and the surface was rough as in common practice. The system is compared to a control steel-reinforced specimen designed using the Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA S6-06). The system is also assessed in terms of ultimate strength, deflection, and stress levels.
4.2 Experimental program

The experimental program comprised a total of five full scale bridge deck specimens (S1 to S5) with the associated precast concrete support girders and the connection detailing (Table 1). Four specimens (S1 to S4) simulated a single span with a center-to-center girder spacing of 1780 mm, while specimen S5 simulated an exterior span with a cantilevered overhang deck of 1070 mm beyond the edge of the outer girder. The idea for S5 was to examine the feasibility and performance of SIP forms in an overhang setting. The following sections provide details of the SIP form profile, material properties, parameters, construction and connection details, test setup and instrumentation.

4.2.1 GFRP SIP form

Figure 4-1 shows the GFRP hat-shaped profile used. The section is mass produced by Creative Pultrusions, Inc., with a proprietary pultruded GFRP layup, comprising E-glass fibres and polyester resin. It is commercially available as a sheet-pile section with pin-and-eye end connections to facilitate interlocking and continuity in the transverse direction. Given the shallow profile of 102 mm of this section along with the interlocking feature, it was deemed quite suitable for investigation as a SIP form for concrete bridge decks. The corrugations provide the necessary stiffness to avoid excessive sagging under the weight of concrete. The narrow width of 457 mm facilitates shipping and handling, where the desired lengths can be cut on site and sections can easily be spliced transversely by sliding the pin-end into the eye-end of another section. Similar profiles of two different thicknesses, namely 3.2 and 4.2 mm, were used in this study.

4.2.2 Material properties

4.2.2.1 GFRP form

The authors carried out independent coupon testing on the GFRP SIP form product according to ASTM D-3039. Figure 4-2 shows a sample stress-strain curve in the longitudinal and transverse directions of the 4.2 mm section, relative to other materials used in this study. The average tensile
strength and modulus were 353 MPa and 24.5 GPa, longitudinally, and were 89 MPa and 9.2 GPa, transversely, for the 3.2 section. For the 4.2 mm section, these values were 344 MPa and 24.5 GPa, longitudinally, and 94 MPa and 9.26 GPa, transversely.

4.2.2.2 Concrete
The concrete compressive strength specified for the support girders and the deck was at least 35 MPa at 28 days, with a maximum aggregate size of 19 mm. The measured cylinder strengths according to ASTM C-39 at time of testing the specimens ranged from 37 and 41 MPa.

4.2.2.3 GFRP rebar
V-Rod number 4 (12.7 mm) GFRP bars were used as top reinforcement in specimens S2 to S5. The manufacturer reported tensile strength and modulus of 786 MPa and 46.3 GPa, respectively (Figure 4-2).

4.2.2.4 Steel rebar
10M (11.3 mm) ‘black’ steel bars were used in the support girders and in the deck of control specimen S1. Tension tests were carried according to ASTM A370 and the measured yield strength was 435 MPa (Figure 4-2).

4.2.3 Test specimens and parameters
A typical specimen of S1 to S4 represents a single span between two girders spaced at 1780 mm. Support girders of AASHTO type III were simulated. The flange width of these girders was 410 mm and the top surface typically has a rough concrete finish with the steel stirrup protruding to provide a connection with the concrete deck. The width of the deck in direction of traffic was 1605 mm. Figure 4-3 shows a typical general view of specimens S2 to S4.

Specimen S1 was a conventional steel reinforced concrete (RC) control deck designed to satisfy all CHBDC specifications for this particular girder spacing. The final design was a 225 mm thick deck with a 0.32 percent steel reinforcement ratio in each direction (i.e. 10M @ 150
mm in each direction), top and bottom, with a clear concrete cover to the outermost layer of 40 mm. To facilitate the bar development over the supporting girders, due to the lack of continuity, the top and bottom steel bars running in the direction normal to the girders were prepared with large end hooks over the girders.

Specimens S2 to S4 used the GFRP SIP form (Figure 4-1), where four modules connected together through the pin-and-eye connections were used in each specimen (Figure 4-3). Concrete was cast on to the forms such that the total thickness to the bottom flange of the GFRP section was 225 mm, similar to specimen S1. In this arrangement, the deck thickness directly under the center of the loading pad was only 125 mm and one pin-and-eye connection was located very close, at 95 mm, from the center of loading (Figure 4-3). Specimens S2 and S3 used the 4.2 mm thick section, while S4 used the 3.2 mm section and no additional bottom reinforcement was used. In specimen S2, adhesive bond was used between the concrete and GFRP form, whereas in S1 and S3 concrete was cast directly on the forms. Top orthogonal layers of 12.7 mm diameter GFRP rebar spaced at 175 mm were provided for shrinkage and temperature crack control and also for negative moment cracking over the supporting girders. This reinforcement was designed according to section 16 of CHBDC with a clear concrete cover to the outermost layer of 35 mm. To facilitate adequate embedment of the top GFRP rebar running in the direction normal to the girders, which can not be bent, the concrete deck was provided with a 200 mm overhang beyond the outer surface of the top flange of the girder, to accommodate the development length of the bars.

Specimen S5 was a single span of 1830 mm with an overhang of 1275 mm (Figure 4-4). The length of the overhang is one half of the girder spacing plus a distance to accommodate the barrier wall. The specimen had a relatively narrow width of 765 mm because it focused mainly on constructability aspects. It comprised two 4.2 mm thickness SIP form sections with one of them having the top flange removed for over all cross-section symmetry (Figure 4-4). One option
was to extend the SIP form of the span into the overhang; however, the protruding stirrups from the upper surface of the supporting girder would necessitate drilling large holes in the form. The other option, used in this study, was to use a separate section for the overhang, which is connected to the SIP form of the span using splices. Five 25x9.5x610 mm GFRP straps were used to connect the top and bottom flanges and the webs of the two SIP form sections on both sides of the exterior girder, using size 14x2” hex head self-taping screws. The straps were designed to support and transfer, to the inner span form, the bending arising from the weight of fresh concrete on the overhang form. The cantilever top reinforcement was designed per unit width as per the requirements of CHBDC for the fully factored service load, which resulted in 7-12.7 mm diameter GFRP rebar.

4.2.4 Fabrication of specimens

Special care was taken in detailing of the supports and simulating accurately the connection between the decks and supporting girders. For specimens S1 to S4, I-shape supports with replaceable top flanges were used (Figure 4-3). The 410 mm wide and 250 mm deep flanges extended the full width of the deck of 1605 mm and were reinforced to conform to the flange of standard AASHTO type III precast girder. This included protruding 10M (11.3 mm) stirrup ends for mechanical anchorage with the concrete deck and a rough surface finish. The flanges were bolted, using integrally cast threaded rods, to 18 mm thick base steel plates anchored to the support concrete bases, after providing a thin layer of plaster at the interface. The supports on either side of the deck were connected using two 25 mm diameter high strength threaded rods, simulating the bracing effect of diaphragms in a bridge.

For specimen S1, conventional wooden formwork and shoring system was prepared for casting the deck. For specimens S2 to S4, a method of installation of the deck was developed based on previous work (Berg et al. 2006, and Reising et al. 2002) which simulates the anticipated actual process of constructing in the field. Foam haunches, 13x13 mm, were glued
using common caulking to the concrete flanges of the two supports at their inner edges (Figure 4-3) (the height of these haunches in practice can be varied to accommodate elevation changes of the deck in the transverse direction of the bridge). The GFRP SIP forms were cut to 1550 mm lengths and connected through the pin-and-eye connections. The assembly was then rested on the foam haunches such that the panel ends extend 90 mm past the inner edges of the support girder on either side. The corrugations of the panels result in voids under the panel crests. These were easily dammed using pre-cut foam inserts (Figure 4-3). The top of each support girder was then grouted underneath the GFRP form, up to the level of the panel trough. This created a high quality bearing surface between the SIP form panel and the girder which is essential for load transfer. After the grout was set, the top GFRP reinforcement was placed, and the concrete deck was cast on to the form. For specimen S2, epoxy adhesive was applied to the GFRP form surface using a brush, like paint, immediately before concrete pouring. This technique was investigated and described in Honickman et al. (2009) as one of the easy-to-apply methods for promoting bond at the interface of FRP SIP forms and concrete.

For specimen S5, the deck overhang GFRP form was connected to the inner span GFRP form by mechanically fastened GFRP splices, as indicated earlier. The assembly was then rested on the 13x13 mm foam haunches glued to the edges of each concrete support, then grouted underneath the form over the supports as described before (Figure 4-4). GFRP top rebar was placed and concrete was then cast on to the forms. The deflection at the end of the overhang under the weight of concrete was 16 mm. The asymmetric GFRP profile and splicing of the form caused some warping during concreting.

4.2.5 Instrumentation
Specimens S1 to S4 were tested under a single load applied at the center of the deck using a standard 510x255x25 mm steel plate over a 12 mm thick neoprene pad, to simulate a standard truck tire (Figure 4-3 and Figure 4-5(a)). A spherical seat was located on top of this pad. A 2000
kN MTS hydraulic actuator was used to apply the load at a rate of loading of 1 mm/min. Specimen S5 was loaded under a full axle loading (i.e. two loads spaced 1830 mm apart according to AASHTO/LRFD specifications), at a rate of 1 mm/min. The load on the overhang was applied at 590 mm from the free end, which is the closest practical location of the tire of the truck to the free end, given the space occupied by the barrier wall and also the external dimensions of the design truck (Figure 4-4 and Figure 4-5(b)).

Deflections were measured using Linear Potentiometers (LPs) at various points along the two axes passing through the center of decks S1 to S4. Strains on top concrete surface and GFRP SIP form were also measured at various points along the same axes using 5 mm electric resistance strain gauges and 100 mm displacement-type position indicators (PIs). Shear strains were also measured in the web of the form. For control specimen S1, strain gauges were used to measure the strain of the bottom steel rebar. LPs were also used to measure any horizontal slip that might occur between the GFRP SIP form and the inner faces of the supporting girders and also between the top GFRP rebar and concrete. For specimen S5, deflection along the length of the over hang was measured using LPs and strains were also measured using strain gauges and PI gauges at the maximum moment location.

4.3 Experimental results and evaluation of performance

4.3.1 Performance criteria

In a field application of a different FRP SIP form system by the Ohio Department of Transportation, performance criteria were established to indicate acceptable performance (Alagusundaramoorthy et al., 2006) and will be used in the current study also. The criteria were based on deflection, strain and shear strength requirements. The deflection was not to exceed span (L)/800 at service load (other DOTs use a more conservative criterion of L/1600, which will also be examined here). The strain in the GFRP form was not to exceed 20 percent of ultimate strain under service load, and 10 percent under dead loads only. Shear requirements stipulated that the strength of the deck should at least match the calculated capacity of a comparable
conventional steel-reinforced concrete deck. For service live load, the CHBDC specifies the use of the standard CL-625 truck for bridge analysis. The maximum half axle load of this design truck plus the appropriate Dynamic Load Allowance (DLA) is 122.5 kN. In the United States, the HS-25 design truck represents an equivalent heavy truck for analysis, with a half axle load plus DLA of 115.6 kN.

### 4.3.2 Experimental results

Figure 4-6 shows the load-deflection performances of all test specimens. Deflections are reported at mid-span of all specimens, except in S5, the deflection reported is directly under the load applied on the cantilever. Table 1 provides a summary of the key test results, including ultimate loads, strength safety factors relative to the service load of 122.5 kN, deflection at service load in comparison to the L/800 and L/1600 limits, and the longitudinal and transverse strains at service load and their percentages of ultimate strain. Figure 4-7 shows the deflection profiles in the longitudinal and transverse directions at service load of 122.5 kN.

Figure 4-6 and Table 1 show that all specimens exhibited an ultimate load well in excess of the design truck half axle load of 122.5 kN, and the factors of safety ranged from 2.4 to 5.67. The deflection at service loads for specimens S1 to S4 ranged from 0.26 to 0.59 mm, well below the L/1600 limit of 1.1 mm. The maximum strains in the GFRP SIP forms did not exceed 0.0003, which is 1.6 percent of the ultimate strain.

All specimens S1 to S4 failed in punching shear, yet Figure 4-6 shows that control specimen S1 failed at a load 16 to 62 percent higher than specimens S2 to S4. It should be noted, however, that a major difference between specimen S1 and the others is that S1 had a uniform thickness of 225 mm including concrete cover, while specimens S2 to S4 had an average depth of 175 mm because of the corrugations. Additionally, the GFRP form configuration and its bond to concrete behave differently from steel reinforcement, particularly near failure as described next. It
is worth noting that slip of the GFRP form relative to the support girders did not exceed 0.01 mm at ultimate.

### 4.3.3 Failure modes

In specimens S1 to S4, negative moment cracks were observed early on, on the upper surface above the girders, due to the restraint conditions provided by the supports. In specimen S1, once punching shear failure occurred at the peak load (Figure 4-8(a)), the load dropped significantly and suddenly (Figure 4-7). This brittle failure is further supported by the fact that the tension steel rebar strain at failure of S1 is only 0.0025, barely yielding (Figure 4-9). On the other hand, in specimens S2 to S4, it is very clear that punching shear does not lead to a sudden loss of strength but the load drops very gradually and the process is associated with large deformability and deflections (Figure 4-7). Figure 4-8 (b to d) shows that punching shear is associated with a seam tearing of the GFRP section, initiating near the support at the junction between the bottom flange and inclined web. This tearing develops gradually into a diagonal tension fracture in the GFRP web, due to the principal tensile strains. The tearing was initiated by a principal strain below the ultimate longitudinal or transverse tensile strain due to the primarily 0-90 fibre architecture.

In specimen S5, the cantilever section failed in flexure by rupture of the GFRP top bars (Figure 4-8(e)) over the exterior support. This is further evident by the tensile strain in the top GFRP bars at failure (Figure 4-10), which approaches the rupture strain reported in Figure 4-2. It should be noted that the width of the specimen was limited to 765 mm, hence flexural failure occurred. This has likely resulted in a conservative strength and a specimen with a larger width would have promoted a higher failure load and less deflections. As indicated earlier, the primary objective of specimen S5 was to examine practical constructability aspects of using the SIP form for overhangs and the 765 mm width was sufficient to demonstrate this aspect.

### 4.3.4 Effect of adhesive bond

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Specimens S2 and S3 were fabricated identically except for the presence of a wet adhesive applied to the GFRP form surface immediately prior to casting. Figure 4-11(a) shows a close-up comparison of load-deflection responses of the two specimens. Initially, before cracking, the stiffness of both specimens was quite similar, up to 176 kN, which is well above the service load of 122.5 kN, where negative moment cracking occurred on the top surface over the girders. It is likely that cracking in the positive moment region followed almost immediately as a result of the loss of stiffness arising from cracking at the fixed ends, leading to an increase of the positive moment. However, the authors had no way of visually verifying the onset of positive moment cracking because the SIP form covered the underside of the deck. This cracking resulted in a reduction of the overall stiffness, which was gradual in specimen S2 due to the excellent bond, but was quite abrupt in specimen S3 (Figure 4-11(a)). Honickman et al (2009) observed that adhesive bond of GFRP SIP flat forms to concrete slabs tested in flexure demonstrated a large network of very well distributed fine cracks that are hardly visible, unlike conventional flexural cracking in RC where discrete cracking occurs at certain spacing. It is believed that in specimen S3, perhaps a single large crack occurred in the positive moment region and the loss of natural adhesion resulted in global debonding, where the SIP form was acting as a tie anchored at both ends. Nonetheless, as indicated earlier, there was no evidence of any slip of the forms from the girders at the ends. Although bond clearly affected the stiffness of the system significantly after cracking, it had a very little effect on the ultimate load, since failure mode in both cases was a punching shear.

Figure 4-11(b) shows the load-strains at the extreme tension fibers of the form in the longitudinal and transverse directions. A sudden increase in longitudinal strain is observed in specimen S2 upon cracking and generally strains in S2 are much larger than in S3 with adhesive bond. The maximum strain at failure did not exceed 0.0045, which is well below the ultimate value of 0.0183.
Figure 4-11(c) shows the load-strains responses in the webs of the forms at the vicinities of the punching shear crack paths. The strain rosette responses are plotted and were used to calculate the principal tensile strain response. Significant difference in responses is observed for specimens S2 and S3, where cracking and debonding is strongly reflected in the response of S3, whereas S2 showed a much smaller principal strains. These principal strains resulted in the diagonal tension fracture of the webs as shown in Figure 4-8(d).

It should be noted that previous work (Honickman et. al., 2009) into bond effects on flat slabs has demonstrated that the use of a bond mechanism should be strongly recommended. In lieu of a wet adhesive system, a bonded aggregate system can achieve similar performance while presenting practical advantages in installation.

4.3.5 Effect of SIP form thickness
Specimens S3 and S4 used GFRP forms of two different thicknesses, 4.2 mm and 3.2 mm, respectively. Neither of these specimens used adhesive bonding at the interface and both employed similar top layers of bi-directional GFRP rebar. Figure 4-12(a) shows the load-deflection responses, while Figure 4-12(b) shows the load-strain responses of both specimens. The initial stiffness of specimen S4 was lower than that of S3 and it did not demonstrate a distinct cracking point. It is possible that S4 may have been already cracked before the test though this is unlikely given the minimal deadweight and short spans of these decks. Stiffness after cracking was similar in both specimens, but continued to decline in S4 until punching shear failure occurred at a load 20 percent lower than S3, which had a 24 percent thicker GFRP form.

4.3.6 Performance of pin and eye connection
Previous work (Dieter et al, 2002 and Nelson and Fam, 2009) has shown that the splice between adjacent GFRP SIP form sections could potentially be a source of weakness in the system and may govern strength. In this study, no failure occurred at the splice. Figure 4-13 shows the load-transverse strain responses for specimens S2, S3 and S4, under the load, just 95 mm away from
the pin-and-eye connection. The maximum transverse strain did not exceed 0.00035, which is less than 3 percent of the ultimate value of the GFRP section in the transverse direction (Figure 4-2).

4.4 Summary and conclusions

In this study, pultruded GFRP corrugated plates, connected together through pin-and-eye connections were investigated as stay-in-place (SIP) forms for concrete decks. The SIP form completely replaced the bottom layer of reinforcement, while a top GFRP mesh was provided. Special attention was given in simulating the details of deck connection to supporting girders. Full-scale deck specimens, including a control deck with conventional steel reinforcement, were cast on concrete supports simulating a girder spacing of 1780 mm. The 410 mm wide supports had a rough surface finish along with protruding steel stirrups to simulate the flange of AASHTO type III girder. Additional cantilevered specimen was tested to examine the feasibility of using SIP forms in deck overhangs at exterior girders. The study also investigated the effects of GFRP plate thickness and bond with concrete on performance. All specimens, except the cantilevered one that failed in flexure, had a punching shear failure while the GFRP pin-and-eye connections remained intact. The system demonstrated excellent performance, with safety factors ranging from 3.5 to 4.9, relative to the half-axle service load, including impact factor, of standard design trucks. Deflections at service were less than span/1600. The system also displayed significant deformability associated with gradual loss of strength beyond punching shear, a major advantage over conventional decks. Adhesive bond improved stiffness but had little effect on strength. Constructability issues are also addressed along with the detailing particular to this system.

Results of this study suggest that this corrugated FRP SIP formwork system is a safe and efficient system for constructing composite concrete bridge decks. They system exhibits high transverse stiffness and meets deflection requirements. It is recommended that this system should see application in a field project in the near future. The most important feature of the system yet to be
investigated is freeze-thaw resistance and associated performance of the concrete-FRP interface. Work on examining this feature is currently being carried out at Queen’s university.

4.5 Acknowledgements

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4.6 References


ASTM A370 “Standard test methods and definitions for mechanical testing of steel products”.


Table 4-1: Summary of test results

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>Parameter / GFRP SIP section</th>
<th>Ultimate load (kN)</th>
<th>Service load (kN)</th>
<th>Ultimate load</th>
<th>Deflection at service load (mm)</th>
<th>Limit L/800 (mm)</th>
<th>Limit L/1600 (mm)</th>
<th>Strain in SIP form at service load 122.5 kN x10^-6</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Control steel-RC</td>
<td>694</td>
<td>CHBDC CL-625, Impact 122.5</td>
<td>94</td>
<td>5.67</td>
<td>4.87</td>
<td>4.32</td>
<td>3.49</td>
</tr>
<tr>
<td>S2</td>
<td>4.2 mm + adhesive</td>
<td>596</td>
<td>4.2 mm, no adhesive</td>
<td>529</td>
<td>0.42</td>
<td>0.32</td>
<td>0.26</td>
<td>0.59</td>
</tr>
<tr>
<td>S3</td>
<td>4.2 mm, no adhesive</td>
<td>529</td>
<td>3.2 mm, no adhesive</td>
<td>427</td>
<td>2.2</td>
<td>4.70</td>
<td>0.26</td>
<td>1.63</td>
</tr>
<tr>
<td>S4</td>
<td>3.2 mm, no adhesive</td>
<td>294</td>
<td>4.2 mm cantilever</td>
<td>294</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>73 (0.61)</td>
</tr>
</tbody>
</table>
Figure 4-1 Corrugated GFRP panels with pin-and-eye connections

Figure 4-2: Stress-strain responses of GFRP corrugated panels relative to other rebar used
Figure 4-3: Typical schematic views and pictures of specimens S2 to S4

Figure 4-4: Specimen S5 with GFRP ties holding SIP overhang form
Figure 4-5: Test Setups

Figure 4-6: Load-deflection responses of test specimens
Figure 4-7: Deflection distribution at service load of 122.5 kN

Figure 4-8: Failure modes
Figure 4-9: Load-strain behavior of control specimen S1

Figure 4-10: Load-maximum longitudinal strains in specimen S5
Figure 4-11: Effect of adhesive bond on responses (S2 vs. S3)
Figure 4-12: Effect of GFRP-SIP form thickness on responses (S3 vs. S4)

Figure 4-13: Load-transverse strain responses near the pin-and-eye connection closest to load for specimens S2 to S4
Chapter 5

Full bridge testing at scale constructed with novel FRP stay-in-place structural forms for concrete deck

5.1 Introduction

Concrete decks are ubiquitous in highway bridge construction, forming the driving surface and also often acting as a structural element of the bridge superstructure. The exposure of bridge decks to direct contact from vehicles, moisture and often chlorides makes them vulnerable to corrosion and deterioration at a higher rate than the rest of the bridge structure. For this reason, Fiber Reinforced Polymer (FRP) composites have been gaining acceptance and popularity in bridge construction. Stay-in-place (SIP) formwork for concrete decks has been proposed but still limited in application. It seeks to reduce labor costs associated with installing conventional disposable formwork and has the potential to act compositely with the concrete slab as flexural reinforcement. Current systems of SIP formwork include precast prestressed concrete panels (Kluge and Sawyer, 1975) and cold rolled corrugated steel panels (Grace et al., 2004). These systems do present disadvantages, such as corrosion and asthetic issues with corrugated steel SIP formwork and difficulty of installation and cost associated with concrete precast panels. Seeking to address these deficiencies, a system was envisioned using FRP SIP formwork to provide the advantages of SIP forms without the constructability and durability drawbacks of conventional systems (Bakeri, 1989). These novel hybrid decks have been investigated as simply supported one way slabs (Hall and Mottram, 1998; Honickman and Fam, 2009), as two way slabs (Dieter et al., 2002; Alagusundaramoorthy et al., 2006) and in the field (Berg et al., 2006; Matta et al., 2006). Consideration has also been given to some boundary conditions of these systems in one

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direction (Nelson and Fam, 2012). There is currently no literature on the structural performance at ultimate limit states of the FRP SIP form system in the context of real boundary conditions pertaining to continuity in both directions as found in a full bridge. Additionally, many parameters related to the FRP SIP system remain to be investigated.

This chapter seeks to evaluate the effect of various parameters of the FRP SIP form system under the most realistic possible boundary conditions of a fully constructed bridge, similar to early work done on conventional bridge decks (Batchelor et al, 1972). The testing program offers opportunities to evaluate performance features not possible with traditional independent limited size deck sections, such as:

1. Effect of restraint on deck strength and stiffness, with particular focus on interior versus exterior deck spans.
2. Effect of the deck edge location and the corresponding effect of the monolithic diaphragms at this location (i.e. location along the span of the bridge).
3. Effect of deck span (i.e. girder spacing) without the corresponding impact on the aspect ratio of the test pad.
4. Effect of girders as semi-fixed support, where the girders deflect and may rotate depending on their proximity to a diaphragm. The effect of any longitudinal compression force in the slab from composite action with the girders is also accounted for.

5.2 Experimental program
The study reported in this article involves the design, construction and testing of a full composite slab on girder bridge at 1:2.75 scale. The full scale bridge was designed for a 12.5 m span according to the CAN/CSA S6-06 bridge design code and incorporates 4 CPCI 1200 (AASHTO Type III equivalent) girders on neoprene bearing pads, cast with a composite concrete deck. An overview of the bridge can be seen in Figure 5-1. Due to the localized nature of the deck failure mode, the bridge was used as a test bed for 15 individual bridge deck tests, independent from one
another in location and design. This section provides details on the experimental program, including specimens, construction, and material properties.

5.2.1 Test specimens and parameters

Figure 5-1(a) shows a top view layout of the test bridge, where 15 individual tests were carried out to study various parameters, while Figure 5-1(b) shows cross-sections. Pads A1-5 and B1-5 have a common center-to-center span of 665 mm (i.e. representing 1830 mm (6 ft.) girder spacing at full scale). Pads C1-5 span is 887 mm center-to-center (i.e. 33% longer span, representing 2440 mm (8 ft.) girder spacing at full scale). Pads B1, B2, C1 and C2 are built using conventional RC construction including a top and bottom grid of steel rebar. All other pads in the bridge are constructed using the novel FRP SIP formwork system. Pads B3, B5, A3 and C3-5 are all constructed using identical detailing. Also, loading of these panels is applied directly over the splice location of the FRP panels. The splice involves an overlap between the flat FRP panels with high-modulus epoxy adhesive bond and self-drilling mechanical fasteners within the overlap. Pad A1 is identical to the above pads except for the addition of adhesive at the concrete-FRP interface to promote full composite action. This is a special adhesive that bonds freshly cast concrete to the FRP surface. The loading in pads A2 and A4 is applied directly over the center of the FRP panel (i.e. at the farthest possible distance from the splices). The splices in pad A2 are bonded as mentioned above, but the splice in A4 does not include any panel-to-panel bonding (i.e. only an overlap). Pad B4 also does not include any panel-to-panel bond; however, loading is applied above the splice location in this pad. Table 1 provides a summary of test matrix showing various parameters, including splice details, which is either a combination of mechanical and adhesive bonding (M+A), or nothing (in practice a bead of caulking).

5.2.2 Materials

The bridge was constructed using two concrete mix designs, GFRP pultruded panels, steel and GFRP reinforcing bars and two types of high modulus epoxy adhesives. Their properties are
described in the following sections. Figure 5-2(a) shows the stress-strain behavior of several of
the constituent materials.

5.2.2.1 GFRP ribbed panels
A commercially available ribbed GFRP panel was used in the construction of the SIP formwork
sections of the bridge deck. It consists of a flat plate with integrated T-shape ribs spaced at 50
mm and is produced by pultrusion using E-glass fibers and a polyester resin. The panel is 40 mm
deep and is produced in 500 mm wide sections which were cut to fit this application. The
laminate architecture is a series of longitudinal, transverse and roving fiber mats with longitudinal
fibers being the dominant orientation. Tensile coupon testing was carried out in both directions
according to ASTM D3039/D3039M (2008). It revealed longitudinal tensile strength of 399 MPa
with a modulus of elasticity of 24.7 GPa, and a transverse tensile strength of 50 MPa with a
modulus of 12.7 GPa. Tensile testing of the web of the ribs was also conducted and resulted in a
tensile strength of 354 MPa and a modulus of 25.1 GPa (Figure 5-2(a)).

5.2.2.2 GFRP reinforcing bars
Commercially available GFRP reinforcing bars were used as top reinforcement for the deck in the
regions where FRP SIP forms were used. All GFRP reinforcing bars were type #2 V-Rod bars.
Manufacturer data reports a guaranteed tensile strength of 784 MPa with a tensile modulus of
elasticity of 46.1 GPa.

5.2.2.3 Steel rebar
6 mm diameter deformed steel wire was used as steel reinforcing bars. Tensile testing was carried
out as per ASTM A370 (2010) and determined a yield stress of 462 MPa with a modulus of
elasticity of 194.5 GPa and ultimate strain of 0.033.

5.2.2.4 Concrete
The concrete mix used in the composite girders was designed for a 45 MPa at 28 days and the exact strength was 46.9 MPa. The mix used in the deck slab was designed for 35 MPa, using high early strength and the measured strength was 40.4 MPa at 90 days (first date of testing). Aggregates in both mixes were of 10 mm maximum size. Concrete was tested as per ASTM C-39 (2010).

5.2.2.5 Epoxy adhesives
Two adhesives were used in the construction of the bridge. When bonding the FRP panel-to-panel splice, 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. The concrete-FRP interface in pad A1 was bonded using Sikadur 32 Hi-Mod, a high modulus, and low viscosity epoxy adhesive with manufacturer reported bond strength of 13.1 MPa.

5.2.3 Design of test bridge
Previous research at full scale (Nelson and Fam, 2012) was used as a basis for establishing the design bridge. A full scale bridge was first designed with a longitudinal span of 12.5 m and a width of 8.7 m, including overhangs. The bridge was designed as a composite slab-on-girder bridge comprising four CPCI 1200 (AASHTO Type III equivalent) girders. It was designed at full scale following standard methods outlined in the CHBDC (CAN/CSA S6-06) for a prestressed composite concrete bridge. Simply supported end conditions were adopted and the standard CL-625 design truck was used along with lane loading. In particular, the bridge deck was designed using the empirical method available in the CHBDC. The following necessary preconditions were met in order to use the method: the slab is of nearly uniform thickness and bounded by the exterior beams, it is composite with the supporting beams, which are themselves parallel, the beams are no more than 4.0m apart (at full scale), and the deck span to depth ratio is less than 18. The diaphragms were designed from standard details in the CHBDC which are approved for use with the empirical method described above. The girders were simply supported on plain
elastomeric pad (PEP) bearings designed as per AASHTO LRFD clause 14.7.6.3. Concrete diaphragms were located at each end of the bridge with no intermediate diaphragms. The diaphragms were integral with the slab and were designed using two methods: Proportioning for the effect of applied wheel load directly on the diaphragm; and as per a standard detail designed to provide additional in-plane restraint to the deck (CAN/CSA S6-06 clause 8.18.6).

Upon designing the full scale bridge, it was scaled geometrically by a factor of 1:2.75 to establish the test bridge. This scale factor was selected based on the available size of FRP SIP formwork panels. Based on which, the test bridge has a span of 4350 mm and a deck width of 3250 mm. For practical reasons, composite steel-concrete girders were used in the test bridge to simulate the full scale prestressed concrete girders. They matched closely to their prestressed counterparts in flexural and torsional stiffness. This was achieved by controlling the size of the concrete cap on the steel girders. The top of the cap was identical in geometry to the top surface of a conventional CPCI 1200 precast girder scaled by the 1:2.75 ratio, while its depth was designed to match the stiffness. Figure 5-1(d) shows a cutaway of the bridge deck indicating the girders and other key components. Stirrup heads were protruded from the concrete girder caps along with surface roughening of the concrete in order to promote composite action with the deck as in typical precast girders. The girders were analyzed at small scale to ensure they remained fully elastic under the maximum anticipated deck punching load in order to avoid progressive damage affecting subsequent tests.

5.2.4 Fabrication of test bridge

Construction of the test bridge was conducted as closely as possible to what would be encountered in practice. Concrete abutments were fabricated and neoprene bearing pads were installed and leveled. The prefabricated girders were then lifted into place. Once the girders were aligned, end diaphragms were cast integrally. Conventional formwork was constructed for pads B1,2 and C1,2 and the deck overhang. The FRP SIP form panels were then set on thin foam
haunches located on the edges of the precast girders (Figure 5-1(b)) and were grouted underneath as per the process established by Nelson and Fam (2012). The top FRP rebar cage was tied into place. Finally the steel rebar in the control region was set and tied. Concrete was placed using a hopper with a handheld vibrator for consolidation, as shown in Figure 5-1(c). Final leveling was accomplished using a laser level and the deck was cured for seven days using wet burlap. Once 28 day strength was achieved, the overhangs and control RC formwork was stripped, a process not necessary for the FRP SIP deck portions.

5.2.5 Test setup and instrumentation

Loading was applied using a 100 ton hydraulic ram through a swivel joint and a tire pad. The pad was designed to simulate the footprint of a design truck tire as per the AASHTO LRFD (2007) and scaled down using the scale factor. It measured 91 mm (traffic direction) by 182 mm (transverse direction) at scale. It was composed of a 12.7 mm thick neoprene pad in contact with the deck, under a thick steel plate. Load was measured using a 445 kN MTS load cell while deflections were measured using up to 11 Linear Potentiometers (LPs) for each pad test. Strains in the FRP and steel components were measured using an extensive array of 120 Ω electric resistance strain gauges. A combination of 5 mm uniaxial gauges and 10 mm biaxial gauges were used. Concrete strains were obtained using several 100 mm 350 Ω Pi gauges (strain gauge based transducers). Figure 5-2(b) illustrates the test setup during the testing of pad B3.

Test instrumentation was generally set up based on a minimum skeleton instrumentation layout, with additional instrumentation monitoring particular locations or responses of interest. For each pad test, this included deflection measurements at regular stations in the traffic direction along the deck span centerline and in the transverse direction under the location of loading. Deflections of interest were usually those of the deck itself without the superstructure included; to accomplish this, LPs stationed at the abutments and under the girders were used to produce a deflected shape for the superstructure and the local deflection of the girders was subtracted from
the measured deck deflections. Unless otherwise mentioned, deflections measured on the deck have been corrected in this manner.

Displacements were measured in a similar fashion for the conventional RC control pads, while strains were measured using strain gauges mounted directly on the rebar cage. Typical layout for the RC decks included longitudinal and transverse strains on the top and bottom mat of reinforcement directly under the load pad.

5.2.6 Establishing service and factored loads at scale

Two methods were used for establishing an equivalent truck load. First, an equivalent nominal punching shear stress method was implemented as per Graddy et al. 2002. Nominal punching shear stresses were calculated as follows:

\[ \nu = \frac{P}{(b_o \cdot d)} \]  

(1)

Where \( \nu \) represents the nominal punching shear stress over the punching cone, \( P \) is the applied tire load, \( b_o \) is the punching shear perimeter and \( d \) is the average shear depth of the section. For this evaluation, a crack angle \( \theta \) of 39 degrees was used as suggested by Fang et al. (1990). Using the service load as the CL-625+I wheel load (CAN/CSA S6-06) of 122.5kN, a nominal service shear stress of 0.344 MPa is achieved. This stress is then applied to the geometry of the scaled deck, resulting in a 18 kN service load.

The second method involves correlating the flexural strains and curvature at service load between full scale and small scale. At the CL-625+I service load, the full scale deck experienced a maximum tensile strain of 255 \( \mu \)e and a curvature of 2.1x10\(^{-6} \) m\(^{-1} \), both measured at the centerline in the direction of primary bending (Nelson and Fam, 2012). At a similar FRP strain, the small scale bridge experiences a load of 24.3 kN. The corresponding curvature is 6.15x10\(^{-6} \) m\(^{-1} \). This curvature is roughly a factor of 2.75 larger than its full scale counterpart due to the geometric scale factor. It is then decided to use the 24.3 kN as the equivalent service load at the
small scale, since it is based on direct strain measurements rather than equivalent shear stress analysis and several assumptions in the first method.

On the other hand, as the failure mode of restrained concrete decks is overwhelmingly in punching shear, it may be more sensible to scale the factored required strength at ultimate limit states using the equivalent shear stress method. Using this scaling, the fully factored (ULS) single wheel load is 30.6kN.

5.3 Experimental results
This section presents test results in terms of a comparison between performances of RC control deck pads and FRP SIP pads, along with the effects of various parameters on performance. Table 1 presents a summary of results, including the peak load, deflections and strains. In general, peak loads were between 5.5 and 8.5 times the established service load level. Also, interior span panels exhibited higher stiffness and strength than exterior span panels.

5.3.1 Performance of RC control decks
A total of four conventional RC pads B1, B2, C1 and C2 were designed as per the CAN/CSA S6-06 empirical method and tested to provide a benchmark for the results. The RC decks were 75 mm thick. Deck B2 represents the most restrained RC deck pad. Figure 5-3(a, b) show the load-deflection and load-strain data for this pad. As expected, the deck’s performance was dominated by flexure until peak load, at which point it failed in punching shear. The characteristic change of slope that is visible around the service load level represents flexural cracking of the concrete slab, above which the flexural stiffness is decreased significantly. After numerous cycles, the test would adopt this cracked slope from the onset of loading and the uncracked portion would not be visible. Yielding of reinforcement occurred before failure only in the bottom mat, transverse direction, with other rebar strains not exceeding 50% of yield strain. Despite this, a considerable permanent deflection was observed in the bridge deck after testing. Failure by punching shear
was obvious by a considerable sudden increase in deflections at stations 2, CC, 4 and QS at the point of peak load.

Pad B2 (along with all B-series pads) had girder-to-girder spacing of 665 mm. Pad C2 was designed in identical fashion but had a span of 887 mm. In specimens B1 and B2, the punching cone perimeter intercepted the girders at their edges while in C2, the cone was entirely contained in the span. The punching cone of pad C2 can be seen in Figure 5-3(c, d). The cone is characterized by radial flexural cracks at the soffit extending from slightly outside the footprint of the loading pad to a diameter of approximately 600 mm at the soffit. Negative flexural cracks occurred on top along the girder edges. The negative moment cracks are characteristic of all pad tests. Figure 5-3(f,g) shows the load-deflection and load-strain responses for pad C2. The longer span of this pad resulted in considerably higher flexural strains and deflections, relative to pad B2. On the other hand, the ultimate load of both specimens was comparable as failure mode was through punching shear. The 8% decline in capacity compared to B2 should be attributed to less restraint in the exterior span of C2. By comparing the transverse curvature at mid-span and at girder face (Figure 5-3(e)) the effect of span length on flexural response is clear. The high apparent negative cracking load of deck C2 is due to the strain gauge’s failure to intercept the initial flexural crack.

5.3.2 General performance of the FRP SIP form system
Test pad B3 represents the most restrained deck pad, being at the center of the bridge. For this reason, it will be used as a reference to consider the general performance of the novel deck system. Figure 5-4(a) shows the load-deflection response of pad B3 at multiple locations. The stiffness and peak load were slightly lower than control RC deck. This is attributed to the 15% smaller thickness of 65 mm, relative to the RC deck. Response of pad B3 was governed by flexure in biaxial bending, up to roughly its peak load, at which point a punching shear failure occurred. Once punching shear failure occurred and the peak load was reached, loading was
stopped in order to avoid spreading damage to adjacent test pads. Figure 5-4(b,c) show the transverse strains and curvature responses, respectively. A negative curvature can be seen at the girders. Strain gauges on the FRP form in the transverse direction registered some tension at the girders before ultimate load, despite the negative curvature at that location. This is consistent with some arching action developing in the slab, with the FRP panel acting as a tie. Figure 5-4(d) shows longitudinal deflection profiles at various load levels. Longitudinal flexure of the slab can be clearly seen, and careful examination shows a small uplift at higher load levels at 1.14 m from the load. Figure 5-4(e) shows longitudinal profiles of bottom panel strains, captured at the same load levels. It can be seen by both the deflection and strain profiles in longitudinal direction that there is a point of contra flexure to the North and South of the load pad, after which the slab is in negative longitudinal flexure. The longitudinal Pi gauge Pi1 registered a small concrete tension prior to punching shear. The bottom FRP longitudinal surface strains L1 and L2 (Figure 5-4(b)) show reversal across the lap splice, indicating progressive peeling of the joint starting at about 100 kN. Similar behavior was reported by Nelson and Fam (2012).

5.3.3 Effect of span length and restraint on FRP SIP form system performance
Pads A3, B3 and C3 were designed to examine the effect of span length and the effect of interior versus exterior deck location. A3 and B3 had a center-to-center span of 665 mm while C3 had a span of 887 mm. A3 and C3 were exterior spans, while B3 was an interior span. Figure 5-5 shows a comparison of the response of these pads. A comparison of A3 and C3 shows that the ultimate capacity of the deck is unaffected by span length punching shear failure governs, however, the stiffness of C3 was lower. It should be noted that below a certain span length the girders will impinge on the punching cone of the slab and artificially increase punching capacity, while very long spans may fail in flexure before punching shear. Of interest is the difference in performance between pads A3 and B3. The post-cracking stiffness for A3 was 34 kN/mm while B3 was 55 kN/mm, a 62% increase. The two deck pads were designed and constructed in identical fashion.
The improved performance of B3 is attributable to its location in the deck. The additional restraint afforded to interior spans increases both the stiffness and the ultimate capacity while the capacity determined by regular single or two span bridge deck tests produces conservative results. Pad B3 was tested before A3 but it is not believed that this was a contributing factor in the difference in performance. Longitudinal strains measured at the splice location indicate that peeling occurred earlier in the longer span C3 and C4 specimens than in A3 and B3. This is likely due to the increased curvature in these specimens.

5.3.4 Effect of FRP splice detailing and location of loading

The panel-to-panel splice is a location of particular interest in FRP SIP form systems. This study employed two variations of panel to panel connections: fully bonded with a combination of mechanical fastening and adhesive bond, and fully unbounded. Each option was tested in a separate pad with the load applied directly above the splice (B3 and B4), or with the load directly centered above the panel half way between splices (A2 and A4). Figure 5-6 compares the performances of the two bonding methods. The response of the slabs before failure is very similar regardless of splice location or detailing (Figure 5-6(a)). This is because only the longitudinal positive moment, which is not the primary bending, is affected by the splice detailing but the dominant bending is in the transverse direction. The punching shear capacity was affected by splice detailing, but only when the loading was centered on the panel. It is suggested that this drop in capacity is due to a steeper shear cone forming with its roots at the panel seams. This phenomenon has been targeted for future research.

The longitudinal strain gauge on pad B3 (Figure 5-6(b)) indicated tension followed by progressive peeling, while the longitudinal gauge on B4 shows very little compressive strain (i.e. lacks tension in longitudinal direction). Transverse strains were very similar regardless of splice detailing; this was also observed in A2 and A4. With full (M+A) bonding of the seam, the punching capacity was approximately the same regardless of the location of applied load. Pads
A3 and A2 are identical apart from the relative location of loading; they failed at a peak load of 149.4 kN and 138.6 kN respectively. Pads B3 and A2 cannot be directly compared because they are located in interior and exterior spans, respectively.

5.3.5 Effect of concrete-FRP interface bonding

The load-deflection responses of pads A1 and A5, which are identical except for the addition of a wet adhesive in A1, are compared in Figure 5-7(a). The adhesive was applied to the horizontal surfaces of the form panel just before casting and cured to create a good bond at the concrete-FRP interface. Previous research (Honickman et al., 2009) has shown that this technique is sufficient to move the interface failure plane into the concrete mortar layer by exceeding the shear strength of the mortar itself. Figure 5-7(a) shows that the ultimate load of A1 was 162.5 kN, 29% higher than A5 (126.4 kN). Figure 5-8(a) compares the bottom panel strains in the transverse direction for pads A1 and A5 and shows that at any given load level; the bonded FRP strain in A1 is larger than that of the unbounded FRP in A5. Figure 5-8(b) shows the load-strain responses at three locations along the span. The figure shows that peeling in A1 began at 130 kN at the centerline and progressed to the quarter span by a load of 134 kN. For comparison, the splice strains of pad A5 are also shown, where peeling can be seen starting at the centerline at 50 kN and reaching quarter span at 90 kN. It is worth noting that the gain in strength due to adhesive bond compensates for the reduction in strength of outer panels. It should also be noted that while applying wet epoxy would be cumbersome in the field, the bonded aggregate method described by Honickman et al (2009) achieves similar performance and can be factory-installed on the panels.

5.3.6 Effect of proximity to diaphragms and deck edges

Table 1 shows that, for the interior span, the ultimate load of B3 pad was 32% higher than B5 end pad. For the outer spans, pads A3, A4 and A5 are identical, apart from their distance from the end of the span, same with pads C3, C4 and C5. Load-deflection results of these pads are shown in
Figure 5-7(a) and confirm that the end of the span represents the weakest location on the deck, despite the presence of the integral diaphragm at that location. The drop in capacity near the diaphragms was apparent in both the FRP SIP formwork pads and in the traditional steel-RC pads. Rather than considering this a decrease in performance at the ends near diaphragms, the anomaly can be considered a bonus in capacity in other locations on the deck, provided by the additional in-plane restraint. This is similar to the divergence in performance between pads A3 and B3 (Figure 5-5(a)). The diaphragms at C5 and B5 were proportioned for the effects of applied wheel load (CAN/CSA S6-06, clause 8.18.6) while additional reinforcement was added to the diaphragm at A5 as per a standard detail provided (CAN/CSA S6/06, Figure 5-8, 4th detail). The peak load of pad A5 was 5% lower than B5 (Table 1) so if this additional reinforcement benefitted the capacity of the slab it was more than offset by the lack of restraint available at the exterior span.

It was anticipated that the rotational fixity provided by the diaphragms would contribute to deck stiffness by approaching a more fixed-fixed support condition. For this reason, girder rotations were measured for C3-5 using LPs placed horizontally at the girder top and bottom (see Figure 5-7(c)). The results support the conclusion that the diaphragms do restrain the girders against rotation. Girders in pad C3 (farthest from diaphragms) show rotations up to 0.05 rad. while the pad C5 girders did not rotate at all. Pad C4 was more restrained than C3, but less than C5. Figure 5-7(b) shows that the rotational fixity provided a small increase in flexural stiffness in the service load range for C5, and an even larger increase for A1 and A5. Despite this, no increase in peak capacity was observed from this phenomenon but to the contrary strength was slightly reduced as indicated earlier. Figure 5-7(d) shows the measured lateral deflections of the girders at C3 and C5 with locations indicated. The plot highlights that while C3 experienced significant rotations, C5 experienced a rigid body translation of its girders apart from each other. This translation was
due to a crack in the diaphragm and suggests that the slab lost lateral restraint, thereby reducing its capacity.

5.4 Conclusions
A novel system of FRP Stay-in-place (SIP) formwork for concrete bridge deck is tested within a full bridge setting, including multiple girders, diaphragms and monolithic connections between components. The 4350x3250 mm footprint of the test bridge represents a 1:2.75 scale of a 12.5 m span full scale bridge. It includes four simulated precast concrete girders, supported at both ends on neoprene bearing pads with integral concrete end diaphragms. The deck is cast compositely on the girders using the FRP SIP forms, except for a control portion constructed using traditional steel reinforcement. Fifteen tests were carried out to investigate the effects of girder spacing, interior versus exterior spans, end- versus middle-sections, loading on versus loading off splices of FRP panels, FRP surface treatment, and splice method. Results show the FRP SIP form system performed quite well relative to control pads and code requirements, exhibiting safety factors between 2.8 and 4.3 over factored loads. Location of test pads had a substantial impact on performance, leading to a difference of up to 32% in capacity, with exterior and edge pads exhibiting the lowest loads. Using adhesive bond at concrete-FRP interface mitigated this reduction. Lack of panel-to-panel splice detailing reduced strength by 21% if loading was between splices, but no reduction occurred when loading was directly above splice.

5.5 Acknowledgements
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5.6 References


Table 5-1: Summary of test results

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Construction</th>
<th>Span length (mm)</th>
<th>Location</th>
<th>Joint detail</th>
<th>Peak load (kN)</th>
<th>Deflection @ 20.9kN (mm)</th>
<th>Peak strain in FRP panel (µε)</th>
<th>Load location</th>
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<td>CL splice</td>
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Note: All peak strains are longitudinal or transverse as indicated.
Figure 5-1: Overview of bridge deck layout and fabrication

Figure 5-2: Mechanical properties of materials and test setup
Figure 5-3: Performance of traditional RC decks
Figure 5-4: General performance of FRP SIP form system
Figure 5-5: Effect of restraint, span and pad location
Figure 5-6: Effect of splice detailing and location of loading
Figure 5-7: Effect of constraining effect of girders and proximity from supports
Figure 5-8: Effect of concrete-FRP interface bonding
Chapter 6

The effects of splices and bond on performance of bridge decks with FRP stay-in-place forms at various boundary conditions

6.1 Introduction

The state of transportation infrastructure in North America has been deteriorating for some time. This has necessitated a substantial investment in research to improve the durability and longevity of new concrete infrastructure. Much of this work has been focused on bridge decks, which are particularly susceptible to deterioration due to their extreme exposures to environmental conditions and, in the North, chloride attack from road salt. Fiber Reinforced Polymer (FRP) composites have been recognized as a promising solution to this problem in all-FRP configurations and as internal reinforcement for bridge decks (Bakis et al. 2003). Recently, FRP has been investigated as a Stay-in-Place (SIP) formwork system for bridge decks, combining the durability advantages of FRPs with the inherent constructability advantages of SIP formwork (Berg et al. 2006, Ringelstetter et al. 2006). Various FRP-SIP geometric configurations, including corrugated plates were explored (Fam and Nelson 2012, and Nelson and Fam 2012). In these systems, various new products with no history of FRP SIP application were introduced. The products offer optimisation in terms of FRP-concrete mechanical interlock and/or more efficient reinforcement ratios. Numerical investigations of composite systems have been conducted, including work on slabs (Cheng and Karbhari 2006) and beams (Foraboschi 2012). At present, available results confirm the structural performance of various configurations of FRP SIP formwork, but do not address various parameterers relating to the detailing of the system. The experimental work related to these systems has largely been focused on testing or modeling?

discrete bridge deck sections, with width-to-span (w/s, see Figure 6-1) aspect ratios typically less than one. These tests can justifiably be expected to produce conservative results as compared to actual decks constructed in the field, due to the discontinuities at the free edges, but there is no literature or guides on this margin of conservatism. This fact has not been a drawback for testing which has predominantly been conducted to validate individual field projects, but is an impediment to research where the goal is to evaluate the performance of the system under field conditions. In addition, testing on FRP SIP form deck system has been conducted almost exclusively with the load applied directly over the seam, presuming that this is a conservative practice. It has been established that the level of in-plane restraint provided to bridge decks impacts their performance (Batchelor et al. 1978), making this a critical parameter. The study described in this chapter addresses these concerns by targeting several parameters using scaled bridge deck specimens. A total of 10 decks were constructed examining the following parameters for decks with FRP-SIP forms, specifically, along with a control steel-reinforced deck:

(a) *The effect of aspect ratio*: a variety of aspect ratios were chosen from narrow to very wide with all other parameters held constant. This isolated the effect of specimen width in discrete deck panel tests and enabled the determination and recommendations of critical aspect ratios.

(b) *The effect of concrete compressive strength*: the series of widths mentioned above were tested at two different concrete strengths. This is an important parameter, given that failure is expected by punching shear and the strength is governed by in-plane restraint.

(c) *The effect of interface bond*: a specimen was constructed using epoxy adhesive between the FRP panel and concrete to achieve composite action and compared to specimens cast on FRP panels without surface treatment.
(d) *The effect of location of load relative to the splices of FRP panels:* identical decks were constructed where the load was applied either directly above the splice or directly above the panel center, half way between splices.

(e) *The effect of rotational fixity at deck supports on the stiffness and capacity:* one specimen was tested on simply supported rollers to compare with specimens monolithically cast with simulated precast concrete girders.

### 6.2 Experimental program

The following sections provide a summary of test matrix, materials, fabrication and test setup and instrumentation.

#### 6.2.1 Test specimens and parameters

In general, the system under investigation represents a novel FRP composite SIP formwork system, as developed at full scale in an earlier study (Nelson and Fam 2012). The system is composed of a ribbed GFRP plate, produced by the pultrusion method, placed between the girders of a slab-on-girder bridge in such a way as to act as permanent formwork and replace the bottom layer of reinforcement. The panels are placed with the ribbed stiffeners transverse to the direction of traffic and panels are joined to one-another at panel to panel splices located at regular intervals in the traffic direction (Figure 6-1). In some cases, surface treatment is provided to the FRP-concrete interface to promote composite action, while in other cases no bonding is provided at the interface.

Table 1 provides a summary of test matrix. As shown in Figure 6-1(a), specimens A1-A3 were constructed with variable slab widths of 286, 604 and 886 mm, representing deck width-to-span aspect ratios of 0.43, 0.91 and 1.33, respectively. Specimen A4 represents a deck tested earlier by the authors in a parallel study. This deck was identical to the system used for specimens A1-A3, except that it accurately represents a real condition deck of a complete bridge constructed over multiple girders of 4350 mm span. The width of 4350 mm was established in a parallel study
based on a scaled down full bridge design. It is added to specimens A1-A3 to represent the case of a virtually infinite width (in this case 4350 mm), giving an aspect ratio of 6.54. Deck A4 has a concrete strength of 40 MPa, comparable to A1-A3, which are 37-46 MPa.

Specimens B1-B3 were similar in design to slabs A1-A3 (Figure 6-1), except that they used a very low strength concrete (17 MPa). Low strength concrete filling may present economic advantages and/or permit the use of low density concrete should structural performance be adequate.

Specimens C1-C3 investigated the effect of FRP-concrete bond, loading location relative to FRP splices and support conditions. Unlike specimens A1-A4 which were cast directly on FRP panels without surface treatment, specimens C1-C3 were constructed using a high modulus epoxy adhesive bond at the FRP-concrete interface in order to promote full composite action. While loading was applied directly over the panel-to-panel splice in C1, C2 was loaded half-way between splices as shown in Figure 6-2. C1 and C2 were both cast monolithically onto scaled-down simulated CPCI 1200 (AASHTO Type III equivalent) girders to simulate field application. In contrast, C3 was cast independently as a flat slab, and supported on rollers at each end during testing, as shown in Figure 6-2. This simulated the conditions under which many deck slabs are currently tested for convenience. Specimens A1 and C1 are identical other than the FRP-concrete interface epoxy, thereby assessing its structural contribution.

Control specimen C4 is a conventional steel-reinforced concrete deck, shown in Figure 6-2, used to compare the FRP SIP system to conventional practice. This deck was designed as per CAN/CSA S6-06 requirement and included a top and bottom orthogonal layers of steel bars, with 0.3% steel reinforcement ratio top and bottom in each direction. The thickness of the FRP SIP form specimens was designed to be equal to the depth to the bottom steel reinforcement of control specimen C4.
All specimens had a 665 mm span center-to-center. This span along with all other dimensions of the deck and supporting girders represent a 1:2.75 scale from a prototype full scale deck designed earlier by the authors to represent a 1830 mm (6 ft.) girder spacing commonly used in practice and supported by CPCI 1200 (AASHTO Type III equivalent) precast concrete girders (Nelson and Fam, 2012).

6.2.2 Materials

6.2.2.1 GFRP form panels
The GFRP panels used as SIP formwork in this study can be seen in Figure 6-3. They consist of a 4.2 mm thick GFRP plate, stiffened by intermittent T-shaped ribs spaced at 50 mm center-to-center. The total panel depth is 40 mm and the rib web and flange are both 4.2 mm thick. The ribs are oriented transverse to traffic direction when installed. Tensile coupon testing was carried out in both directions according to ASTM D3039/D3039M (2008). It revealed linear longitudinal stress-strain curve with a longitudinal tensile strength of 399 MPa and a modulus of elasticity of 24.7 GPa. In the transverse direction, response was slightly nonlinear with a tensile strength of 50 MPa, an initial modulus of 12.7 GPa and a rupture strain of 0.0088. Tensile testing of the web of the ribs was similar to the flat part and resulted in a tensile strength of 354 MPa and a modulus of 25 GPa.

6.2.2.2 GFRP reinforcing bars
Commercially available GFRP reinforcing bars were used as top reinforcement for the deck in the regions where FRP SIP forms were used. All GFRP reinforcing bars were type #2 V-Rod bars. Manufacturer data reports a guaranteed tensile strength of 784 MPa with a tensile modulus of elasticity of 46.1 GPa.

6.2.2.3 Concrete
Two concrete strengths were used in this study. Normal strength concrete was designed for 35 MPa and exhibited compressive strength between 37 and 46 MPa on the day of testing according to ASTM C-39 (2010). The low strength concrete had a 28 day strength of 15 MPa and 17 MPa at time of testing. Both concrete mixes had 10 mm coarse aggregates.

6.2.2.4 Steel rebar

6 mm diameter deformed steel wire was used as steel reinforcing bars. Tensile testing was carried out as per ASTM A370 (2010) and determined a yield stress of 462 MPa with a modulus of elasticity of 194.5 GPa and ultimate strain of 0.033.

6.2.3 Fabrication of deck specimens

Decks were constructed in a manner similar to the process that would be used in the field. Initially, support girders were constructed by preparing concrete caps of the same shape and design as a scaled top flange of CPCI 1200 precast concrete girders. This concrete supports included protruding stirrups and a roughened top surface to promote a monolithic connection (Figure 6-1 and Figure 6-2). The stirrups were located at 55 mm spacing, representing a full scale spacing of 152 mm. For ease of fabrication, the concrete caps were cast on steel beams with steel studs. Once the concrete caps had cured, SIP FRP formwork panels were installed. Styrofoam haunches, 7 mm tall were placed along the edge of the girder cap. In the field, these haunches could be installed in various sizes to accommodate cross-falls and girder camber. The FRP SIP panels were then placed directly on the haunches in such a way that they extended past the haunch over the girder by 30 mm. The space created under the panel ends was then filled with a non-shrink grout to promote an ideal bearing surface between FRP panels and girder (Figure 6-3(a)). During setting, the panels were bonded together at the panel-to-panel splice location using an 18 mm wide lap splice and high modulus epoxy. Once the grout and epoxy had set, mechanical fasteners of self-drilling #14 screws spaced at 152 mm were installed along the lap splice, in addition to the adhesive. For all decks, a grid of GFRP rebar was tied and placed as top
reinforcement for crack control and negative moment region over the support. This top reinforcing cage can be placed directly on the T-ribs in this system, eliminating the necessity of chairs (Figure 6-3(a)). In order to provide sufficient anchorage of the GFRP top rebar, deck overhangs equal to the development length of the bars were provided beyond the support girders (Figure 6-1 and Figure 6-2). For decks A1-A4 and B1-B3 concrete was then poured directly onto the FRP formwork. In specimens C1-C3, a special epoxy was painted onto the horizontal surfaces of the FRP panels approximately, 30 minutes before pouring. This epoxy is specially for bonding to freshly cast concrete.

6.2.4 Test setup and instrumentation
The test setup used in this study can be seen in Figure 6-3(b). Loading was applied using a 100 ton hydraulic ram and measured using a 445 kN load cell. To simulate the design truck, loading was applied through a simulated tire load pad, scaled to 91x182 mm (traffic direction by transverse direction), with a 12.7 mm elastomeric pad in contact with the concrete surface. Deflections were measured directly under the load pad on the slab soffit and at several stations along the centerline of the span. Deflection was also measured at the quarter span location and directly above the girder. All deflection measurements were accomplished using 100 mm linear potentiometers. Strains on the FRP panel were measured in a variety of locations using 5 mm and 10 mm electric resistance strain gauges, while top surface concrete strains were measured using 100 mm displacement type Pi gauge transducers. During testing, the support girders were connected using tie rods to restrain them against rotation.

6.3 Experimental results
The following sections present the effects of various parameters studied for the FRP SIP forms on performance, namely, deck width, concrete strength, adhesive bond, splice location, and support condition. For convention, in this section, the longitudinal axis refers to the axis of primary
bending (normal to the supporting girders) and the transverse axis refers to the direction perpendicular to the longitudinal axis (parallel to the supporting girders).

6.3.1 Performance of novel deck system
Figure 6-4 compares the load-deflection responses of the control reinforced-concrete deck C4 (which is designed according to CAN/CSA S6-06 code requirements for steel reinforced concrete decks) and specimen A2. Specimen A2 represents a novel FRP SIP formwork deck of identical width and span, and a comparable concrete strength. It is observed that the ultimate strengths are quite similar though the stiffness of C4 is higher. The higher pre-cracking stiffness of C4 is as a result of the nature of steel reinforced concrete: While both decks were designed with identical depth to reinforcement, for the control deck this meant a larger overall slab depth due to bottom cover which is not present in the novel deck system. Although both decks failed in punching shear, specimen A2 showed a much better ductility beyond punching shear, due to the progressive tearing of the mechanical fasteners through the GFRP plate, whereas in C4, the load drops suddenly upon reaching the punching shear. Additional pseudo-ductility is available from the FRP SIP formwork system due to the presence of the T-shaped ribs, which provide considerable dowel action and shear reinforcement to the slab after the formation of shear cracks. The punching shear crack in specimen C4 was clearly visible after testing on the top slab surface, but on the bottom surface it was intercepted by the slab edges, suggesting a wider specimen would be required to fully mobilize the punching shear capacity.

6.3.2 Effect of deck aspect ratio
Figure 6-4 compares the load-deflection responses of decks A1, A2, A3 and A4 with width-to-span (w/s) aspect ratios of 0.43, 0.91, 1.33 and 6.54, respectively. Specimen A4 was tested as part of a full 4 girder bridge in the laboratory, and is expected to represent the actual upper bound performance of an in-place bridge deck. The peak loads generally corresponds to the shear failure loads, however, the load remains stable or reduces very gradually thereafter. Figure 6-5 shows
the variation of both the punching shear loads and peak loads for specimens A1 to A4 as well as specimens B1 to B3 of lower concrete strength, with the deck aspect ratio. Figure 6-5 shows that the strength envelope can be characterized by three distinct regions. At (w/s) ratio up to 0.6, the slab is excessively narrow and behavior is governed by one-way shear failure as shown in Figure 6-6(a and b); and the strength is much lower than the in-place performance (A4). In this range, the slab performs in a fundamentally different way as in-place bridge decks and results have little relevance in simulating system performance. From (w/s) ratio of 0.6 to 1.6, the slab responds increasingly in 2 way bending, failure is governed by two-way punching shear and the strength increases gradually as (w/s) increases. Within this range the punching cone (failure surface) intercepts the free side boundaries of the deck. For this reason, increases in the width also increase the surface area of the punching cone and the strength is increased as a result, as shown in Figure 6-6(c) for A2 and B2. Beyond (w/s) ratio of 1.6, the strength becomes constant, independent of the (w/s) ratio as the failure surface perimeter becomes fully contained within the deck as in A3, B3 and A4. It is then recommended for laboratory tests of bridge decks to provide a width of at least 1.6 the span for accurate simulation of a continuous deck.

Deflections and strain were monitored in the transverse direction to investigate the extent of transverse bending before failure, which is expected to be affected by slab widths. Figure 6-7(a, b and c) show the transverse deflection, strain and curvature profiles, respectively, for decks A1-4. Curvature was calculated from the strain profile obtained from a top Pi gauge and bottom strain gauge. After normalizing the Pi gauge reading to the top of the slab, the slope of the resulting strain profile (curvature) was calculated using the slab depth. The centerline slab deflections decreased with increasing deck width, as the load was shared in the longitudinal direction. If the slab is idealized as a 1-way strip as in the flexural method of slab design, the effective width of this strip would increase along with the deck width, resulting in higher flexural stiffness. Despite this, it was observed that the transverse curvature was similar for all specimens.
The maximum recorded GFRP transverse strain at peak loads was about 0.003, while the ultimate transverse strain is 0.0039. Transverse compressive strain in concrete approached -0.003 but no flexural failure was observed, all specimens failed in shear. The unusual curvature performance of A3 at high loads is due to anomalous readings in the contributing strain gauge (see Figure 6-7b). It is suspected that the gauge may have partially delaminated from the FRP surface.

6.3.3 Impact of concrete strength
Decks A1-A3 and B1-B3 were identical in all aspects, except concrete strength. Figure 6-8 compares the load-deflection responses of A1-A3 to their counterparts B1-B3 decks. Also marked on all curves the point at which shear failure occurred. It is evident from Figure 6-8 that the onset of shear failure is predominantly dependent on curvature (or deflection) as opposed to concrete strength or deck width. This was also reported by others (Muttoni 2008). Clearly the A1-A3 specimens consistently show higher peak loads due to the higher concrete strength. For the widest specimen A3, the maximum strength was about 21% higher than B1. Of particular interest was the post-shear failure load sustained by the lower strength specimens. Decks B1 and B2 in particular exhibited a peak load of 42% and 11%, respectively, greater than the load at shear failure. This increase can be attributed to the substantial dowel action and transverse reinforcement offered by the formwork panel ribs. In higher strength slabs, the combined resistance of the ribs intercepting the punching cone is less than the concrete punching shear capacity of the deck. On the other hand, the lower shear strength of the concrete in specimens B1-B3 meant that the T shaped ribs had a considerable impact on the final capacity of the slab. Due to the low elastic stiffness of the GFRP panels, it took considerable deflection to engage the full shear capacity of the GFRP ribs (13.6 mm for B1 and 14.6 mm for B2). Fracturing of the T ribs was discovered upon inspecting decks A1-A3 and B1 after testing, supporting this conclusion.

6.3.4 Effect of concrete-GFRP interface bond
Figure 6-9 compares the load-deflection and load-strain responses of specimens A2 and C1,
which are identical in all aspects, except bond, where C1 has an adhesive bond between concrete and GFRP. In a previous study (Honickman et al. 2009), this technique was found to eliminate shear slippage at the interface until shear failure of the concrete slab the result being full composite action of the slab. Techniques have been developed to analyze systems numerically where composite action is not achieved (Foraboschi 2009, Foraboschi 2013). Figure 6-9(a) shows that both ultimate strength and elastic stiffness increased in bonded specimen C1, by 30% and 73%, respectively. Bonding increases system performance due to the composite action provided and the dispersion of cracks from the considerable bonded reinforcement ratio provided by the panels, which increases initial stiffness and punching shear capacity. It is noted from Figure 6-9(a) that the increase in stiffness and capacity occurs regardless of the support conditions, with specimen C3 showing substantially increased stiffness with simply supported boundary conditions. The increase in capacity is indeed sufficient to replicated the performance of an in-place deck section (as simulated by specimen A4). Without interface bonding, specimen C3 would be significantly weaker due to the presence of interface slip between the concrete and FRP in the absence of proper end restraints.

Figure 6-9(b) shows that the maximum GFRP longitudinal strains achieved in decks C1 and A2 are 0.0037 and 0.0024, which are much lower than the 0.016 rupture strain of the GFRP. The transverse strains in both decks approach about 0.004, well below the transverse rupture strain of 0.0085. The mechanical response of the deck system at ultimate load is significantly different depending on the presence of composite action. Where composite action is available (as in specimen C1), a strut and tie mechanism forms within the slab, with the panel acting as a tension tie. When composite action is not available, the slab is instead restrained by its boundary conditions and develops arching action. This response is available only if the girders are restrained by diaphragms or other methods (Oliva et al. 2007).

This conclusion is supported by strain measurements. The longitudinal strain response
reverse direction such that, for panel A2 without adhesive, it drops essentially to zero as arching
action develops with external restraint applied by the girders. The panels of deck C1 continue to
carry considerable longitudinal tension strain as a result of the strut and tie system. Finally,
bonding of concrete around the splice location is suspected to increase peeling resistance by
restraining the lap splice from warping under tension.

6.3.5 Effect of FRP splice location relative to load
A parallel study by the authors explored the effect of loading on FRP SIP form splice versus
loading half-way between splices in none composite decks without any surface treatment or
adhesive bond between concrete and GFRP. The study showed that loading directly above the
splice was not the critical case. In fact it showed about 7% higher strength than the other case. In
this study specimens C1 (loaded above splice) and C2 (loaded between splices) are compared for
the same effect but in composite decks with adhesive bond between concrete and GFRP. Figure
6-10(a) compares the load-deflection responses of C1 and C2, while Figure 6-10(b) compares
their load-strain responses based on the strain gauge layout shown in Figure 6-10(c). Figure
6-10(a) shows that deck C1 (loaded above splice) resisted 158 kN while C2 (loaded between
splices) exhibited an increase of 27%, 201 kN. This trend is different from that observed in
unbonded none-composite decks. Figure 6-10(a) also shows that stiffness during the initial stage
was not significantly affected. Figure 6-10(b) shows that the splice of in C1 begins to delaminate
at a maximum GFRP transverse strain of about 0.004, well below the transverse rupture strain of
0.0088, as indicated by the strain reversal (gauge 1T2 in Figure 6-10(b)) and this limits the
overall capacity of the deck. On the other hand, the splice in deck C2 is not at the maximum
strain location (i.e. not at mid-span), therefore, it reaches the same delamination transverse strain
of about 0.004 (gauges 2T2 and 2T4 in Figure 6-10(b)) but at a higher load. At that peak load, the
mid-span transverse strain in C2 (gauge 2T3) approaches the GFRP transverse failure strain of
0.0088 but splice failure occurs first as shown in Figure 6-10(c) and hence load drops.
6.3.6 Effect of rotational fixity at support

Bridge deck testing is often conducted with simply supported boundary conditions, while in reality the deck is monolithically connected to girders. This parameter was examined by comparing monolithically cast specimen C1 and roller-supported specimen C3. The rollers were positioned such that their centerline was located in the middle of the grouted panel bearing surface in the monolithic deck C1. The load-deflection performance of C3 can be seen in comparison to C1 in Figure 6-9. As expected, the simple support conditions resulted in higher deflections during the initial loading phase. Rotational fixity at the girders was monitored by measuring the deflection at the end of the free overhang. The tangent angle was then calculated at the girder location based on the assumption that the overhang rotated as a rigid body. At 100 kN, rotation in specimen C1 was $2.5 \times 10^{-3}$ rad. compared to $7.16 \times 10^{-3}$ rad. for slab C3. The rotational fixity is also illustrated by measured strains. Strains were measured on the top surface of the slab over the edge of the interior girder using a 100 mm PI gauge (Figure 6-11, location in Figure 6-2).

In specimen C1 the gauge intercepted a negative moment crack at 50 kN, whereas virtually no strain was measured at that location in C3, as expected for a simply supported slab. Longitudinal panel strains showed slight increases in panel C3 as a result of its reduced stiffness. Transverse FRP strains were slightly larger in C1 (1T2 in Figure 6-10), compared to C3 (Figure 6-11) at any given load but similar trend of transverse strains indicating splice delamination at 134 kN in C3.

The response of decks C1 and C3 can be explained by basic mechanics. During the initial phases of loading, rotational fixity at the supports reduces deflections by allowing double curvature in the slab. After cracking however, the stiffness of both slabs becomes comparable. Even though a negative moment region exists near the girders in C1, after cracking the flexural stiffness (as a result of the second moment of area) of the slab in negative moment is very small relative to its positive moment stiffness due to the low reinforcement ratio provided by the top bars. This is to be expected as the top reinforcement layer is not designed as flexural reinforcement but for shrinkage and crack control.
6.4 Conclusions
An investigation is conducted on a novel system of Fiber Reinforced Polymer (FRP) Stay in Place (SIP) structural formwork for concrete bridge decks. The deck system is composed of FRP composite ribbed panels, spanning between girders and acting as both permanent formwork and bottom slab reinforcement. The testing, conducted at 1:2.75 scale, consisted of ten bridge deck sections and examined several critical parameters, namely: varying of the width (w) of the deck specimens relative to their spans (s), and varying interface bond condition, concrete strength and loading location on the deck. It was shown that as the width (w) is increased relative to span (s), the performance of the deck approaches the actual built performance. An aspect ratio (w/s) of greater than 1.6 was found to adequately represent performance and avoid overly conservative results produced at lower aspect ratios. Varying concrete strength from 17 MPa to 42 MPa in identical decks resulted in 20% increased capacity but did not influence stiffness. Applying adhesive bond at FRP-concrete interface to create a fully composite section increased the deck strength and initial stiffness by 30% and 73%, respectively. In decks with adhesive bond, loading directly above the FRP splice resulted in a 20% lower strength than loading half-way between splices. This is an opposite trend to that observed in decks without adhesive bond.

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6.6 References


Figure 6-1: Layout of specimens C1-4

Figure 6-2: Layout of specimens A1-3 and B1-3

Figure 6-3: Construction and testing of deck
Figure 6-4: Peak load performance of slabs A1-4 and B1-3

Figure 6-5: Load deflection performance of A1-4, B1-3
Figure 6-6: Failure modes of decks B1 (a, b), B2 (c) and B3 (d)
Figure 6-7: Transverse bending in specimens A1-4
Figure 6-8: Effect of composite action on deck response

Figure 6-9: Effect of concrete strength on performance
Figure 6-10: Location of loading
Figure 6-11: Strains resulting from varying support conditions
Chapter 7

Splices of FRP stay-in-place structural forms in concrete bridge decks

7.1 Introduction

Fiber Reinforced Polymer (FRP) composites have seen wide use as reinforcement in concrete structures, particularly bridges. Their key advantage has traditionally been their superior durability relative to mild steel reinforcement. From this perspective, the longevity of infrastructure can be increased substantially, particularly when exposed to harsh environmental conditions. FRPs are also versatile in that they can be used to produce structural shapes which can then be used on their own, or as stay-in-place (SIP) forms for concrete structures (Bakeri (1989), Hall and Motram (1998)). The resulting system, generally referred to as FRP SIP formwork, combines the advantages of SIP formwork (ease and speed of construction) with those of FRPs (durability and longevity). Due to its relatively lower cost and wide availability, Glass FRP (GFRP) is the most common composite used in these systems. While a variety of different designs have been proposed or investigated, FRP SIP formwork systems all have several common features. They all involve a formwork product which is generally an off-the-shelf or custom pultruded FRP section. The product, referred to herein as the panel, tends to be a plate stiffened with ribs of various cross sections in one direction. These panels are produced in continuous lengths and cut to the desired length. In the field, these panels are placed spanning between bridge girders one next to the other and spliced. The bridge deck systems include a top layer of reinforcing bars for continuity over girders and for thermal and shrinkage crack control.

The system has been investigated in various configurations experimentally and numerically (Nelson et al. (2013a)) but generally speaking, investigations have been conducted with a target design in mind as an exercise in validation (e.g. Hanus et al (2009)). There are

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several critical features of the SIP formwork system which merit investigation. One in particular is the panel-to-panel splice. Joints between individual form panels are unavoidable and techniques for addressing this location have been somewhat not clearly specified in the past. Another related feature is the width of the panels themselves. Again, the FRP SIP formwork system has been implemented with a variety of panel widths, mostly reflecting the products available commercially. This parameter, however, affects the frequency of panel-to-panel splices and should could impacts their performance. This chapter investigates the critical panel-to-panel splice in depth with the following goals:

1) Determine an optimal configuration for the panel-to-panel splice, such that the splice is able to transfer the maximum load from one panel to the next, looking into mechanical and adhesive bond.

2) Investigate this splice in the context of a complete bridge deck section and compare its performance to the simplest possible splice detail in which panels are overlapped without any sort of mechanical or adhesive bond at the overlap.

3) Examine the spacing of splices by comparing bridge decks constructed using different widths of panels and determine the effect of panel width on deck performance.

### 7.2 Experimental program

The FRP SIP form examined in this study is a common pultruded GFRP ribbed plate with T-shaped stiffeners running in one direction as shown in the schematic in Figure 7-1(a). As an SIP form, the panel spans the girders such that the ribs are running in a direction normal to traffic. The panels are available in finite widths; therefore, they need to be spliced. The general idea is to overlap the horizontal plates but with four possible options: (a) just the overlap (i.e. one panel resting on the other with no bond), (b) adhesive bond at overlap, (c) mechanical screws at overlap, or (d) combined adhesive and screws.
This study comprises two distinct but related experimental phases. The first consists of 23 uniaxial coupon-like lap splice specimens. These test specimens were used to evaluate numerous splice configurations in direct tension. The second phase includes the construction and testing of seven bridge decks using various splice configurations selected based on results from the first phase. All deck specimens were constructed at a (1:2.75) scale. The following sections provide details of the materials, design, construction and testing in these phases.

7.2.1 Materials

7.2.1.1 GFRP form panels
A single type of pultruded GFRP panel was used as FRP SIP formwork in this study. The panel is a 4.2 mm thick GFRP plate stiffened by intermittent T-shaped ribs spaced at 50 mm center-to-center, running in one direction (Figure 7-1(a)). The total panel depth is 40 mm and the rib web and flange are both 4.2 mm thick. Tensile testing revealed longitudinal and transverse strengths of 399 and 50 MPa, respectively, with a longitudinal and transverse elastic modulus of 24.7 and 12.7 GPa, respectively.

7.2.1.2 GFRP reinforcing bars
GFRP reinforcing bar of 6.35 mm diameter was used as top reinforcement for all deck specimens. Manufacturer data reports a guaranteed tensile strength of 784 MPa with a tensile modulus of elasticity of 46.1 GPa.

7.2.1.3 Concrete
All bridge decks in the study were cast from the same batch of concrete, specified at 35 MPa with 10 mm maximum aggregate size. The concrete exhibited an average compressive strength of 36 MPa at time of deck testing.

7.2.2 Phase I: Auxiliary splice coupon tests
The objective of this phase was to evaluate numerous possible splice configurations for the GFRP SIP form panels. This included a combination of viscous epoxy adhesive bonding and/or mechanical fastening using self-drilling hex-head screws. To support this objective, test specimens were designed to isolate the panel-to-panel splice location. This design, seen in Figure 7-1, allows uniaxial tension to be applied to the splice. Table 1 lists the entire test matrix of 23 specimens tested in this phase. Parameters under investigation include screw spacing, screw diameter, presence of screws only, presence of epoxy only, combined epoxy and screws, and presence of concrete at the splice. Specimen designation in Table 1 indicates specimens with splice adhesive bond by “A” and provides details of mechanical bond, including screw diameter and spacing. For example S3D12 has three fasteners (screws), spaced at 33 mm and D12 refers to screw size (#12, which is 4 mm diameter). The suffix “C” refers to the presence of concrete filling. The goal of this phase was to establish an optimal splice configuration for further study in bridge deck testing (Phase II).

7.2.2.1 Fabrication
Each specimen comprised two FRP sections, both 100 mm wide, with 18 mm length overlap (4.2 times plate thickness). To avoid eccentricity at the grip location during uniaxial tests, a pair of tabs were cut and bonded to each specimen at the ends (Figure 7-1(a)). At the overlap splice region, the surface was cleaned using acetone to ensure sound bond surface for adhesive. A 2 mm thick Sikadur 30 high-modulus, high-strength epoxy paste was then applied uniformly to the bonding interface. The manufacturer reported a 7-Day tensile strength of 24.8 MPa for the adhesive. The specimens were set to cure for 3 days. If specimens required mechanical fasteners, the drilling process was conducted after the specimens were fully cured. Each self-drilling hex-head screw was installed perpendicular to the surface of the GFRP plate, at the mid-length of the 18 mm overlap as seen in Figure 7-1(a). The screw diameters used were 2.74, 3.12, 3.5, 4.00 and 4.59 mm. For the specimens with concrete, formwork was set after all fastening was completed.
Concrete blocks of equal thickness to the bridge decks tested in Phase (II) were cast on the splice region as shown in Figure 7-1(b).

7.2.2.2 Test setup and instrumentation
The test setup for this phase can be seen in Figure 7-1. Uniaxial testing was conducted in an Instron 8800 testing machine with hydraulic grips. The machine was operated in stroke control to observe residual performance, and load was monitored through an integrated ±250 kN load cell. Joint slippage was monitored in the form of relative panel-to-panel motion by a 25 mm linear potentiometer (LP). The LP could not be installed over a gauge length exactly equal to the lap splice but covered a longer gauge length. However, in all cases, elongation of the panels themselves were subtracted from the LP measurement based on the panel strains measured using electric resistance strain gauges.

7.2.3 Phase II: Bridge deck testing with different splice details
The objectives of this phase are: (a) examine the overall all structural performance and strength of a complete bridge deck system using a selected splice configuration and compare performance and strength with the case of splicing by just overlapping the forms without using any type of mechanical or adhesive bond, and (b) examine the effect of panel width by varying the panel-to-panel (or splice-to-splice) spacing, using a specific splice detail. In particular, it is sought to determine whether the panel spacing can affect the punching shear crack and ultimate capacity in these systems.

These objectives were evaluated using bridge deck sections scaled down at (1:2.75) from those tested by Nelson and Fam (2013) at full scale. The full scale decks were 185 mm thick and a girder-to-girder span of 1829 mm (6 ft. nominal) was used. Top reinforcement consisted of a mat of GFRP rebar designed as per the empirical method of the Canadian Highway Bridge Design Code (CHBDC, CSA S6-06). The FRP panels were rested 75 mm over the edges of precast concrete support girders, over small hunches and were grouted underneath to form
complete contact between the SIP form and girder top. The deck was then cast monolithically one the AASHTO Type III girders which had protruding stirrups. The FRP plates were commercially available sections with end lips that overlap 50 mm, which scales down to the 18 mm lap used in the current study at a 1:2.75 scale.

In the current study, the scaled deck thickness is 65 mm and the girder spacing is 665 mm. The deck included a 275 mm overhang from each side beyond the center of the support girders, in order to accommodate the development length of the top GFRP rebar (Figure 7-2(a)). The scaled width of the support girder is 145 mm. The width of the deck in the direction parallel to traffic is determined as 960 mm. This is based on a width/span aspect ratio of 1.44, which exceeds the minimum aspect ratio of 1.33 established by Nelson et al (2013b) that is required to simulate the real case of a continuous deck.

Three of the seven deck specimens (120-N, 300-N and 500-N) had no bond of any sort (mechanical or adhesive) at the overlap splices but had variable GFRP plate widths of 123 mm, 311 mm, and 468 mm, respectively, which is essentially the splice spacing, shown in Figure 7-2(b). Another three specimens (120-O, 300-O and 500-O) were quite similar to the previous specimens, except that they had the optimal splice configuration, namely, adhesively bonded splice with 4.59 mm screws spaced at 33 mm. Additional specimen (300-OG) was similar to (300-O), except that the concrete deck was adhesively bonded to the GFRP SIP forms by coating the forms with a thin layer of a special adhesive that bonds to freshly cast concrete.

7.2.3.1 Fabrication
The GFRP ribbed panels were first cut with a table saw to their final dimensions. The panels in group “O” (bonded splices) were then bonded with epoxy adhesive over the 18 mm overlap after proper surface preparation. Following curing of epoxy, the self-drilling fasteners were placed within the splice along a line located at the middle of the 18 mm overlap. The GFRP plates were then placed in a special wooden form to accommodate the 145 mm wide simulated part of the
concrete Type III AASHTO girders and the deck overhangs (Figure 7-2(a)). The simulated girder included typical steel reinforcement and stirrups protruding from the girder top and embedded into the deck as in common practice. The GFRP panels were extended 30 mm on top of the concrete girder to simulate actual practice (75 mm at full scale). The top cage of 6.35 mm orthogonal GFRP rebar spaced at 120 mm was then placed in position. Vertical conduits were placed through the girders to facilitate anchoring the deck to a fixed steel fixture using threaded rods passing through the conduits. The steel fixture is reusable and designed to provide the typical restraints of bridge decks in actual setting, as will be discussed next. Concrete was then placed on the GFRP forms directly, except in 300-OG specimen, in which the SIP form was coated with a special adhesive 30-45 minutes before casting to provide bond to concrete.

7.2.3.2 Test setup and instrumentation
Figure 7-2(a) shows the reusable steel frame. It comprises two stiff steel HSS sections tied together using 30 mm diameter high strength threaded rods. Short vertical plates are welded to the top flange of the HSS sections to restrain the deck laterally. Vertical, 19 mm diameter, threaded rods welded and protruded from the HSS flanges. They fit through the conduits in the concrete beam of the deck. During setup, each individual deck was grouted into place on top of the HSS steel sections using high strength non-shrink grout. Steel channel sections were then used to clamp the deck from top using the threaded rods, with a thin layer of plaster placed between the channel and deck surface as shown in Figure 7-3, which shows the complete assembly of the deck setup. The steel frame is designed to allow the deck to achieve performance similar to that of actual in-place bridge decks in exterior spans. It provides restrains for the deck at the girder location in two critical degrees of freedom: rotation (moment restraint) and lateral expansion (acting as a tension tie). Similar restraints have been employed by other researchers in the past when testing bridge decks (Mufti et al. (1993)).
Load was applied to the specimens at the center using a hydraulic ram through a scaled AASHTO tire pad (181x90 mm steel plate over a thin neoprene pad) (Figure 7-3). The load was applied over the middle of a GFRP panel as opposed to above a splice. This in fact was found to be the more critical arrangement and the weaker condition in previous testing by Nelson et al (2013b) because the crack of punching shear is inclined at an angle from the loading point. Load was measured with a 453 kN load cell. The slabs were instrumented with strain gauges in the longitudinal and transverse directions; including gauges at the splice location. Deflections were measured using linear potentiometers.

7.3 Experimental results
The following section reports the results and discussion for both experimental phases.

7.3.1 Phase I: Auxiliary splice coupon tests
Table 1 provides a summary of peak loads and residual loads after adhesive bond delamination (in specimens with combined adhesive and mechanical fasteners).

7.3.1.1 Effect of bond mechanism at splice
The general performance of various splice bond methods can be seen in Figure 7-4, which shows the load-slip responses of specimens S1D14-A (with a single 4.59 mm screw (i.e. spacing of 100 mm) and adhesive bond “A+M”), S1D14 (with a single 4.59 mm screw only) and S0 (with adhesive bond only). The performance of the adhesive-only specimen is characterized by a low-slip development of the peak load, followed by a sudden loss of the total load once bond failure occurred. Bond failure was triggered by inter-laminar shear failure within the GFRP panel itself (see Figure 7-5 top), with the epoxy remaining intact. The right axis in Figure 7-4 shows the effective uniaxial tensile stress in the plate adjacent to the splice. The adhesive-only splice capacity was generally in the range of 30-35 MPa, representing 60-70% of the unidirectional tensile strength of the GFRP plate. In contrast, the mechanical fastener specimen showed a very
soft response (Figure 7-4) as the fastener requires a considerable slip to become engaged. The fastener reaches a peak load governed by either bearing or bearing and direct tension (Figure 7-5 middle). At peak load, the stress developed in the GFRP plate is only about 7 MPa then the behavior shows slow degradation in capacity. The combination of adhesive and mechanical fastener shows a response that represents almost superposition of the two individual responses (Figure 7-4). Initially, a stiff response governed by the adhesive can be seen. After delamination, the mechanical fastener becomes engaged and provides what will be referred to herein as residual capacity. The residual capacity is consistently lower than the peak load.

7.3.1.2 Effect of fastener spacing for a given diameter
As established above, the presence of adhesive bonding has a profound effect on splice response. Figure 7-6 shows the results of two series of tests, both examining a variety of 4.59 mm fastener spacing from 25 to 100 mm center-to-center. One series included adhesive bonding of the splice while the other is not. For the series with adhesive bond and fasteners, it can be seen that the peak load (dominated by the adhesive) is relatively constant regardless of fastener spacing, though the 33 mm spacing showed the highest capacity. Any capacity lost by reduced bond area because of the holes seems to be compensated by early capacity from the fasteners. The residual capacity (governed by the fasteners) is clearly a function of number of fasteners because of the dominant bearing failure mode. The capacity from 1, 2 and 3 fasteners was 3.5, 6.5 and 9.7 kN, respectively. Around a spacing of 33 mm (3 fasteners), failure mode transitioned to a combination bearing-direct tension failure mode (Figure 7-5 bottom). For this reason, the residual capacity at 25 mm spacing (4 fasteners) is lower than 3 fasteners (Figure 7-6). As a result of this trend, it is suggested to consider the optimal spacing of this diameter of fastener (4.59 mm) as 33 mm. At this spacing, the splice maintains a residual capacity 67% of the peak capacity. The second series without adhesive follows a similar trend for peak load as the residual capacity trend
of the adhesive series. At the optimal spacing of 33 mm, splice without adhesive is able to develop 22.7 MPa of tension in the panel, corresponding to 45% of its uniaxial tensile capacity.

7.3.1.3 Effect of fastener diameter for a constant and variable spacing
The size and number of mechanical fasteners used has an impact on the performance of the splice. Figure 7-7 illustrates the variation of the two parameters and the resulting capacity of the splice. Figure 7-7(left), the number of fasteners (and thus fastener spacing) are held constant at 3, while the diameter is varied and no adhesive is used. It can be seen that reducing the diameter of the fasteners only results in a small decrease in capacity, with fasteners 27% smaller, only reducing capacity by 10%. Figure 7-7(right) shows the result of maintaining roughly a constant cross sectional screw bearing area but increasing the number of fasteners. In this way, 6 smaller fasteners should theoretically provide comparable bearing capacity as 3 larger ones of double the diameters. However, results show a significant decrease in capacity with increasing numbers of fasteners. The smallest screw diameter (2.74 mm), proved challenging to implement. The screw was not of sufficient length to fully engage the bearing failure mode. Instead, the screw failed progressively in a pull-out manner. The threads on these fasteners did not project sufficiently from the far panel face to allow for mechanical gripping of the threads in the event of slippage. It is recommended that any mechanical fastener selected for this application be of sufficient length such that at least two threads protrude from the far panel face. An image of the 2.74 mm screw slippage failure mode can be seen in Figure 7-8(a), in comparison to the typical tilting of the larger fasteners, under one way shear after adhesive bond fails (Figure 7-8(b)).

7.3.1.4 Effect of concrete in the vicinity of splice
In addition to a small benefit from pre-cracked tensile capacity, concrete filling in the splice region significantly increases both the adhesive and mechanical capacity of the splice as can be seen by comparing S3D14-A-C and S3D14-C to their counterparts without concrete. S3D14-A-C with adhesive increased in capacity by 54% (14.2 to 21.9 kN) and S3D14-C without adhesive
increased in strength by 39% (9.7 to 13.6 kN). In the case of a splice without concrete, the panel is free to warp in the splice vicinity, which creates a combination of tensile and shear stress in the bond region (Figure 7-8(b)). By adding concrete, warping is prevented (Figure 7-8(c)) and only shear stresses are applied to the bond region, resulting in an improved performance. Despite concrete only being present on one side of the splice, it effectively prevents warping and peeling.

7.3.1.5 Strain distribution
The primary link between the auxiliary tests conducted here and splice performance in the actual bridge deck is expected to be strain. In particular, the strain measured directly at the splice location (SG1 at the middle of the 18 mm overlap). Figure 7-9 shows the load-strain responses for specimens S3D14-A (with fasteners and adhesive) and S0 (with adhesive only). The response of SG1 shows the pronounced effect of warping at the splice location as the strain reverses to compression. Further away, 50 mm from the splice (SG2 and SG3 on both sides of the plate), reflect the effect of some eccentricity in the system.

Figure 7-10 shows the load-strain responses at the splice location (SG1) for various specimens of different bond systems, namely S0 (adhesive only), S3D14 (fasteners only), S3D14-A (fasteners and adhesive), S3D14-C (fasteners only with concrete) and S3D14-A-C (fasteners and adhesive with concrete). It is very clear that the severity of warping, as reflected by strains, is maximum for S3D14 and reduces somewhat by adding adhesive (S0 and S3D14-A). When concrete is added to S3D14, no compression develops, suggesting significant reduction in warping until near failure. When concrete is added to S3D14-A, strain becomes tensile all the way to failure, suggesting elimination of warping. Extending these observations to bridge deck specimens, we can infer that pronounced tensile strains at the splice indicates intact adhesive, while a reversal in strains indicates peeling of the splice at that location.

7.3.2 Bridge deck testing with different splice details
Table 2 provides a summary of the ultimate loads, deflection at peak load and stiffness before and after cracking. Figure 7-11 shows the load-deflection responses of all deck specimens. There are several observations which can be drawn as discussed in the following sections.

7.3.2.1 Effect of panel width for unbonded form splice
The GFRP panel width has a considerable impact on stiffness and ultimate capacity when no joining (group “N”) is provided at the splice. The peak loads for 120-N, 300-N and 500-N specimens are 136, 155 and 178 kN, respectively. Failure mode in all three specimens was punching shear. The T-ribs of the GFRP deck panels can act as shear reinforcement in the primary flexural direction, but the flat part of the plate is also engaged in dowel action in the weak (transverse) direction. In this way, each panel seam (when not bonded in any way) provides a loss in the dowel action. Shear cracks will preferentially terminate at this splice location so small panel widths may force steep crack angles and lower capacities. It is therefore recommended that the largest practical panel width be used in constructing bridges using this system, particularly where no splice bonding will be employed.

Stiffness before failure is also affected by the panel width. The 120-N panel width allowed for substantial crack formation nearly directly below the loading pad while the widest panel 500-N had seams far enough from mid-span, hence the diagonal crack reaches the soffit far from mid-span. As indicated earlier, Nelson et al (2013) have shown that the case of loading directly over the seam is not as critical as loading over the mid-width of panel. Although the 500-N panel is the widest panel used in this study, it is suspected that larger widths would not produce significant gains in capacity. The reason being this panel width corresponds to about 16 degrees angle for the diagonal punching shear crack, if it spreads all the way to the splice, which is a very shallow angle.

7.3.2.2 Effect of panel width for bonded form splice
In contrast, the panels employing optimal splice detailing (group “O”) performed similarly for all panel widths (Figure 7-11). Specimens 120-O, 300-O, and 500-O exhibited peak loads of 176, 166 and 169 kN, respectively, and exhibited similar stiffness. This result confirms that using optimal splice detailing has a significant effect on deck performance, particularly when using narrow panel widths.

7.3.2.3 Effect of splice bond for a given panel width
At wide enough GFRP SIP form as in specimens 500-N and 500-O, the panel splice bond has no effect on strength and stiffness, while at small widths as in 120-N and 120-O, reducing the panel width by 4 times, resulted in about 23% reduction in deck capacity. Figure 7-12 reports strain measurements across the splice locations for specimen 120-N and 120-O. Measurements indicated very small compressive strains at the splice location in 120-N, supporting the assumption that the splices are largely ineffective in this unbonded configuration. In contrast, significant tensile strains accumulated at the splices of specimen 120-O. At around 100 to 120 kN, the splices began to peel, indicated by a reversal in strains. Unlike splice coupon tests in Phase I under direct tension, peak load of the deck is not governed by first splice debonding, at one location. Instead, debonding occurs progressively starting from under the loading pad and gradually propagating outwards along the splice line. This is best illustrated by comparing readings from SG2 (at mid span) and SG3 (at ¼ span). In SG3, strain reversal and hence peeling initiated at higher load than SG2.

7.3.2.4 Effect of bond at interface of panel and concrete
Figure 7-11 shows the load-deflection response of specimen 300-OG with adhesive bond between concrete and SIP form and specimen 300-O without this bond. Both specimens have bonded splice “O”. It is clear that the interface bond enhanced flexural stiffness and ultimate load, by 27% in this case.
7.3.2.5 Performance of decks relative to equivalent design service load

The performance of the full scale decks tested by Nelson and Fam (2013) was assessed relative to the 122.5 kN design half-axle load of the CL-625 design truck of the CHBDC, with maximum dynamic allowance. Nelson et al (2013b) conducted rigorous experimental scaling analysis and established that at 1:2.75 scale, the equivalent service load is 24.3 kN. Figure 7-11 shows this service load level relative to the overall performance. It is clear that all deck specimens performed similarly at this low load level. The ultimate loads were well higher than service load by a factor ranging from 5.7 to 8.8, depending on bond mechanism.

7.3.2.6 Failure modes

All bridge deck sections failed in punching shear. This was noted during testing by the formation of a clear punching shear crack around the perimeter of the load pad. Figure 7-13 shows the failure modes of specimens 120-N, 120-O and 300-O. The top view of specimen 120-N (Figure 7-13(a)) is typical of punching shear failure in all slabs. Negative moment cracks (left and right) formed early in the test on the top surface along the girders as a result of the end fixity of the deck. This was later followed by a well-developed punching shear crack at peak load. These observations were consistent throughout all the seven deck specimens.

Observations of the soffit during and after the test identified differences between the various configurations. Figure 7-13(b) shows the underside of specimen 120-N during testing just after peak load. A large relative slip was observed between adjacent panels (both horizontal and vertical relative movements), particularly near the load location. A much more pronounced vertical gap was observed in specimen 300-N. In specimen 500-N, a barely observable gap formed only after peak load. The closely spaced splices of specimen 120-N fell within the maximum positive moment zone of the slab and slipped laterally relative to each other when put into tension. Further vertical slip occurred as a result of punching shear. The splices of specimen 300-N do not fall within the maximum positive moment zone and therefore did not slip.
significantly before failure. In this specimen, the splices and the punching shear crack coincided and as a result the entire shear deformation was observed at the location of the splice. The splices of specimen 500-N were located outside both the positive moment zone and the punching cone. This also explains the similar response of specimens 500-N and 500-O.

In contrast, specimens 120-O, 300-O and 500-O did not exhibit large relative deformations at the splices. Delamination was observed in the form of a small crack in the adhesive (Figure 7-13(c)) in specimen 120-O but the mechanical fasteners limited the relative slip. Specimen 300-O exhibited a unique failure mode, where a direct shearing of the form panel took place just adjacent to the seam (Figure 7-13(d)). This shearing occurred at the peak load supporting the fact that the panels contribute some dowel action to punching shear resistance of the slabs.

### 7.4 Summary and conclusions

This experimental study focused on the detailing and spacing of splices of FRP stay-in-place structural forms used in concrete bridge decks. The ribbed FRP panels which span the gap between girders are of typically of limited width, and therefore are spliced by overlapping. A total of 23 auxiliary lap splice tension tests addressed various bond systems including adhesive and/or mechanical fasteners of various diameters and spacing. Seven scaled bridge deck systems were built with FRP panels of various widths, some with bonded splices and some with no bond at all at the overlap region. The study showed that using narrow FRP plates of small width-to-deck thickness (b/t) ratio significantly reduces deck capacity when no splice bond is provided. The reason being, narrow panels may force punching shear crack to terminate at a splice, at a steeper angle, thereby reducing punching shear capacity. The deck with b/t=1.9 exhibited a 24% lower capacity than that with b/t=7.2. In bonded-splice decks, plate width has no effect on strength. Generally, ultimate failure loads ranged from 5.7 to 8.8 times the equivalent service load. For an overlap splice of a length equal to 4.3 times plate thickness, the optimal splice was by combined adhesive and fasteners of diameter equal to plate thickness, spaced at 1.8 times the overlap length.
It developed 68% of plate tensile strength, whereas fasteners alone developed only 14-45% strength depending on their diameter and spacing.

7.5 References


Table 7-1: Test matrix and results of tension splice tests in phase I

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Screw Ø (mm)</th>
<th>CC Spacing (mm)</th>
<th>Peak capacity (adhesive, kN)</th>
<th>Residual capacity/screw capacity (kN)</th>
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</thead>
<tbody>
<tr>
<td>S3D12-A / S3D12</td>
<td>4.00</td>
<td>33.3</td>
<td>11.45</td>
<td>7.28 / 9.27</td>
</tr>
<tr>
<td>S3D10-A / S3D12</td>
<td>3.50</td>
<td>33.3</td>
<td>12.03</td>
<td>7.60 / 8.55</td>
</tr>
<tr>
<td>S3D8-A (3) / S3D8</td>
<td>3.12</td>
<td>33.3</td>
<td>12.71</td>
<td>6.61 / 8.76</td>
</tr>
<tr>
<td>S1D14-A / S1D14</td>
<td>4.59</td>
<td>100</td>
<td>13.71</td>
<td>4.23 / 3.54</td>
</tr>
<tr>
<td>S2D14-A / S2D14</td>
<td>4.59</td>
<td>50</td>
<td>12.62</td>
<td>6.40 / 6.48</td>
</tr>
<tr>
<td>S3D14-A / S3D14</td>
<td>4.59</td>
<td>33.3</td>
<td>14.18</td>
<td>9.55 / 9.73</td>
</tr>
<tr>
<td>S4D14-A / S4D14</td>
<td>4.59</td>
<td>25</td>
<td>13.38</td>
<td>7.60 / 9.93</td>
</tr>
<tr>
<td>S0</td>
<td>NA</td>
<td>NA</td>
<td>12.83</td>
<td>NA</td>
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Table 7-2: Test results of deck specimens in phase II

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak load (kN)</th>
<th>Defl. @ punching (mm)</th>
<th>Pre-crack stiffness (kN/mm)</th>
<th>Post-crack stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>120-N</td>
<td>136.0</td>
<td>6.5</td>
<td>85.12</td>
<td>28.66</td>
</tr>
<tr>
<td>300-N</td>
<td>155.2</td>
<td>4.6</td>
<td>87.89</td>
<td>35.86</td>
</tr>
<tr>
<td>500-N</td>
<td>177.8</td>
<td>4.8</td>
<td>86.78</td>
<td>43.20</td>
</tr>
<tr>
<td>120-O</td>
<td>176.3</td>
<td>5.4</td>
<td>82.75</td>
<td>42.60</td>
</tr>
<tr>
<td>300-O</td>
<td>165.7</td>
<td>5.3</td>
<td>83.56</td>
<td>40.75</td>
</tr>
<tr>
<td>500-O</td>
<td>168.6</td>
<td>4.4</td>
<td>100.13</td>
<td>44.03</td>
</tr>
<tr>
<td>300-OG</td>
<td>211.1</td>
<td>5.1</td>
<td>87.38</td>
<td>58.88</td>
</tr>
</tbody>
</table>
Figure 7-1: Tension splice tests in phase 1: a) without and b) with concrete infill

Figure 7-2: Bridge deck specimens in phase II: a) support system and b) cross sections showing different panel widths and splices
Figure 7-3: View of test setup

Figure 7-4: General load-slip performance of various splicing details
Figure 7-5: Failure modes of lap splices of various bonding configurations
Figure 7-6: Peak and residual capacities at various fastener spacing

Figure 7-7: Effect of fastener diameter on residual capacity
Figure 7-8: Illustration of fastener failure modes with and without concrete

Figure 7-9: Load-strain results measured at various locations on the auxiliary specimens
Figure 7-10: Load-strain responses at splice locations for various specimens

Figure 7-11: Load-deflection responses in bridge decks tested in Phase 2
Figure 7-12: Strain measurements at various locations on specimens 120-N and 120-O

Figure 7-13: Failure modes: a) punching shear from top surface, b) unbonded splice opening from underneath, c) adhesive delamination of splice, d) direct shear failure of panel
Chapter 8

Modeling of flexural behavior and punching shear of concrete bridge decks with FRP stay-in-place forms using the theory of plates

8.1 Introduction

Over the recent past, considerable research has been conducted on the performance of glass fiber reinforced polymer (GFRP) stay-in-place (SIP) structural formwork systems for concrete bridge decks. These systems combine the concept of permanent formwork with GFRP materials, resulting in both rapid construction and more durability than conventional steel reinforced concrete bridge decks. The system has been investigated most thoroughly in the context of composite concrete bridge decks of slab-on-girder type bridges. Numerous experimental investigations of this type of system have been published in recent years (Alagusundaramoorthy et al. (2006), Deiter (2002), Fam and Nelson (2012) and Nelson and Fam (2013a)). These decks have been found to fail overwhelmingly in punching shear due to the nature of loading and slab geometry. Figure 8-1 shows selected configurations of FRP SIP forms, namely flat plate with T-ribs and corrugated plates, which are the sections tested in the experimental work used to evaluate the model in this study. Figure 8-1 also shows the typical configuration of boundary conditions and pictures of sample bridges tested with FRP SIP forms.

Punching shear of concrete slabs without transverse reinforcement has been studied for some time. These slabs, which may or may not be restrained, are common in many types of construction including buildings and bridge decks. Early relevant modeling studies on punching shear of flat slabs were conducted by Kinnunen and Nylander (1960) for isotropic and anisotropic steel reinforcement. Following these principles, modeling was extended to restrained bridge...
decks by Batchelor et al. (1978) using modifications of the Kinnunen and Nylander (1960) model. More recently, punching shear modelling of slabs has been conducted by Muttoni (2008), and Mufti and Newhook (1998), with the former dealing mostly with flat building slabs and the latter considering restrained concrete bridge decks. All of this work was executed in polar coordinates, and was based on a geometric model similar to yield line theory, where radial and circumferential cracks isolate discrete portions of concrete which then undergo solid body rotations and translations. In the case of Muttoni (2008), moment-curvature responses of slabs were integrated along the radial axis to produce an accurate deflected shape in symmetrical problems. Results from this deflected shape are then used to activate punching shear failure criteria. This type of deflected shape is not ideal in the consideration of slab-on-girder bridge decks, which are commonly highly asymmetric in contrast to flat building slabs.

In this chapter, the authors present a model that predicts the full load-deflection response, including the punching shear capacity, of bridge decks with FRP SIP structural forms for slab-on-girder type bridges. First, a deflection field is produced by the model based on the theory of plates and shells (Timoshenko and Woinowski-Krieger (1959)) which accounts for the considerable asymmetry and anisotropy present in the deck. Concrete nonlinearity and cracking are also incorporated into the plate theory. Next, a punching shear failure criterion is applied to the response in order to establish the ultimate capacity of the deck. The chapter first considers the analytical foundation of the model, followed by a validation of the model against a large set of test data, and finally reports the results of a parametric study examining parameters not investigated experimentally.

8.2 Model development
This section explains the analytical basis and development of the model. The model can be said to have three principal features, which are described in the following sections: (1) Determination of moment-curvature performance of slabs in the primary directions, in positive and negative
bending, (2) Calculation of the deflected shape of the slab at a given load, and (3) Application of a punching shear failure criterion to determine the ultimate capacity. Items 1 and 2 together aim at developing incrementally a continuous load-deflection response of the deck, which is terminated at an ultimate load established from a failure criterion, either flexural or punching shear (item 3). Although flexural failure is systematically checked, usually it does not govern in such deck slabs.

8.2.1 Moment-curvature response

In order to calculate the global performance of a slab, its moment-curvature response is calculated in both principle directions (x and y) for positive and negative moments. In particular, the curvature-stiffness performance is sought for the slabs, for use in the plate theory (described later) to establish the deflections. As the slabs are continuous, a unit width of slab is used in calculating its flexural response.

A layer-by-layer strain compatibility approach was used in this study. Figure 8-2 illustrates the mechanics of the model. Similar to the model employed by Honickman (2008) for simply supported beams with FRP SIP forms, the process follows several basic steps. First, the slab is discretized into a number of layers along its depth, with the number being large enough so as individual layers present negligible flexural stiffness. In practice, increasing the number of layers beyond 500 had no discernible effect on moment curvature response, so this value was used consistently. Each layer possesses mechanical and geometric properties appropriate to the material in that layer (which may include a combination of concrete and FRP within the layer). Materials are modeled using the constitutive properties outlined in the next section. In order to model the geometry of the stiffened plate about its strong axis, an effective flange and web width are used to represent the properties of the plate smeared uniformly across the section. In the case of the corrugated FRP plate system, the weak (traffic) direction is modeled using an effective
uniform thickness representing the minimum slab thickness (given that the slab is variable in thickness because of corrugations).

Where the positive moment response is sought, a small increment of tensile strain is applied to the bottom layer. For each increment of strain, the program uses an iterative method to determine the location of the neutral axis. The strain profile is established during each iteration, and the forces from each layer are summed. If the sum of the forces in the axial direction of the slab is not equal to zero, the neutral axis is shifted in the appropriate direction and an additional iteration is performed. When the sum of the forces becomes zero, the neutral axis has been determined. The program then calculates the moment resistance of the section directly from the layer forces. The curvature of the section is also calculated from the slope of the strain profile. Figure 8-3 shows sample moment-curvature responses for one of the specimens used to evaluate the model (FA1 in Table 1). The section’s flexural stiffness (D) at a given loading level is then calculated as the secant slope of the established moment-curvature response.

The above algorithm is successful in developing the moment-curvature response of a composite system, where there is no relative slip between the various components. This is generally the case in conventional reinforced concrete and in FRP SIP systems where mechanical or adhesive bonding is provided at the concrete-FRP interface, which is the case for some of the slabs modeled herein and used for validating the model. In the case where no interface bonding was present (i.e. concrete was cast onto the FRP forms without any mechanical or adhesive bond for simple and easy construction) as in some of the decks tested and used for validating the model, the following modified approach was developed to accommodate for slip.

Based on work related to composite slabs employing steel corrugated decking (Patrick and Bridge (1994)) a factor δ was introduced to allow for slip between the form element and the concrete using the following formula (note that the amount of slip, reflected by a shift in strain profile, is a linear function of the cross section’s curvature (Figure 8-2)).
\[ \varepsilon_{sl} = \delta (h - h_r + h_p) \psi \]

In this equation, \( \varepsilon_{sl} \) denotes the strain discontinuity reflecting the longitudinal slip between the concrete section and the FRP form. The other variables used in the expression are indicated in Figure 8-2. Note that FRP SIP forms of various configurations generally have ribs or corrugations in one direction only (normal to the traffic direction), which provide excellent mechanical interlock in the other (weak) panel direction. For this reason, slip was only employed in calculating response in the strong panel direction, with the orthogonal direction being fully composite. In applying the slip factor, two neutral axes are created (Figure 8-2). As the curvature for both sections remains the same, it is calculated in the regular fashion. Moments can also be calculated in the regular fashion by summing forces about an arbitrary reference.

During execution of the analysis, \( \delta \) is a user inputted constant between 0 and 1, where 0 describes full composite action, and 1.0 describes zero interaction. Despite the fact that the panel surface of GFRP SIP forms are generally smooth, there exists substantial interaction between concrete and FRP even without bonding. This interaction stems from both mechanical interlock at a micro-level and the compatibility condition at the girders (i.e. at termination ends of the FRP SIP forms embedded in concrete above girders). For this study, the factor \( \delta \) was calibrated from test data of decks without composite bonding and a value of 0.2 was found to represent best all non-composite deck slabs. This is shown in Figure 8-4, which represents the experimental versus analytical stiffness of the deck slabs in terms of the slope of the load-deflection responses between 25% and 75% of ultimate load, using a value of 0.2 for \( \delta \).

**8.2.2 Constitutive relationships**

8.2.2.1 FRP SIP forms or top rebar

A linear stress-strain relationship was used for FRP panel components in tension and compression.
where $f_{frp}$, $E_{frp}$ and $e_{frp}$ are the stress, Young’s modulus and strain, respectively. FRP bars were treated as linear in tension, with negligible compression capacity. Where possible, FRP materials data was gleaned from coupon testing. Otherwise, it was obtained from the manufacturer’s reported data. Material failure of the FRP did not govern the failure of deck specimens under any circumstances (apart from very localized phenomena) so using manufacturers data for strengths is not judged to be a problem.

8.2.2.2 Concrete in compression
The model uses the concrete stress-strain relationship from Collins and Mitchell (1997) as follows:

$$f_c = f_c' \left\{ \frac{n(e_c/e_c')}{n - 1 + (e_c/e_c')^n k} \right\} \text{ for } f_c < 0$$

where: $n = 0.8 + f_c'/17$, $k = 0.67 + f_c'/62$, $f_c'$ is the concrete compressive strength, $e_c$ is the strain in the concrete at stress $f_c$ and $e_c'$ is the concrete strain at a stress $f_c'$. The concrete ultimate strain was selected as 0.003, beyond which the concrete carries zero stress.

8.2.2.3 Concrete in tension
Until cracking, a linear elastic relationship was used employing the concrete’s elastic modulus as calculated from CAN/CSA-A23.3-04:

$$E_c = \left( 3300 \sqrt{f_c'} + 6900 \right) \left( \gamma_c/2300 \right)^{1.5}$$

where $\gamma_c$ is the concrete density in kg/m$^3$. Following this:

$$f_c = E_c e_c \text{ for } 0 \leq f_c \leq f_r$$

Beyond the rupture modulus, a tension stiffening relationship was implemented as per Collins and Mitchell (1997). This relationship attributes some tensile capacity to the concrete based on smearing of cracks around reinforcement. In this smearing region, concrete stress is calculated using the following formula:
\[ f_{c} = \frac{\alpha_{1} \alpha_{2} f_{r}}{1 + \sqrt{500(\varepsilon_{c} - \varepsilon_{r})}} \text{ for } f_{c} > f_{r} \]

Factor \( \alpha_{1} \) accounts for reinforcement bond characteristics and is set to 1.0 for composite arrangements and 0.7 for non-composite decks. \( \alpha_{2} \) is 1.0 for monotonic static loading.

Tension stiffening is localized around reinforcement in conventional reinforced concrete, generally between 0 and 7.5 times the bar diameter. In this case, a uniform value of 3.75 times the panel thickness was chosen in all cases to determine the tension stiffening effective region (Honickman (2008)). It should be noted that due to the geometry of the ribbed FRP sections, this region encompasses the entire area of concrete in tension. In the case of the corrugated FRP profile, it forms a discrete region along the panel, with all other concrete beyond modulus of rupture carrying zero stress.

### 8.2.3 Deflected shape

The basic geometry and loading considered by the model can be seen in Figure 8-1(a). Many features of this geometry are not easily accommodated by the polar coordinate systems used in past studies. Of particular interest are the rectangular loading pad, the partially fixed or free boundary conditions (only singly symmetric) and the anisotropic nature of the FRP SIP form reinforcement.

The foundation of the model is based on the concept of thin plates simply supported and subjected to loads normal to their plane. Several assumptions are implied by this formulation: that shear deflections are negligible compared to flexural deformations, that normal stresses transverse to the plate can be disregarded and that none of the materials undergo plastic deformation. It has been shown that a general solution to this problem is the Levy solution (Timoshenko and Woinowski-Krieger (1959)). The general equation of the deflection surface (for an isotropic plate) is:
\[
\frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 y}{\partial y^4} = \frac{q}{D}
\]

where \( w \) is the plate deflection, \( q \) is the applied loading pressure over the entire surface, \( D \) is flexural stiffness and \( x \) and \( y \) are coordinates of a point on plate surface. The solution of Equation 7 takes the following form:

\[
w = \sum_{m=1}^{\infty} Y_m \sin \frac{m\pi x}{a}
\]

where \( m \) is index of summation, \( a \) is the clear span of slab between girders in x-direction (Figure 8-1(a)), \( x \) is the location measured from one end in the span x-direction and \( Y_m \) is a function of location in y direction. \( Y_m \) depends on boundary conditions of the plate. In the following sections, expressions for \( w \) will be developed to address three specific boundary conditions in the y-(traffic) direction, as shown in Figure 8-1(a). In all cases, the boundary condition in the other (x-) direction could be hinged or fixed (in practice decks are monolithically fixed into supporting girders, however, some experimental cases used in validation of the model were simply supported):

(a) Infinitely long plate \((b/a > 4)\), where \( b \) is the width of the slab in y-(traffic) direction: this case represents typical decks in practice supported by long girders (the full bridge in Figure 8-1(b)).

(b) Finite width plate \((b/a < 4)\) with ends in y-direction supported by integral diaphragms connecting girders: this case usually represents the end region of a bridge.

(c) Finite width plate \((b/a < 4)\) with free edges in y-direction: while this is not the case in practice, most experimental tests on bridge decks are carried out on specimens of finite width portions of a bridge (the bridge section in Figure 8-1(b)).

Another feature of bridge decks is that loading \( q \) is applied over a limited area through a rectangular loading pad of dimensions \( u \) in x-direction and \( v \) in y-direction. Also, the loading pad could be located at any point at a distance \( \xi \) from the edge of the girder, along x-direction.
However, for most experimental validation cases $\zeta$ was equal to $a/2$ (i.e. mid-span for maximum deflections and stresses).

8.2.3.1 Infinitely long plate
The case of an infinitely long plate in $y$-direction can be considered a symmetric problem. In order to accommodate fixity at the supports in $x$-direction, the total deflection at any location can be considered as:

$$w(x, y) = w_1 + w_2$$  \hspace{1cm} 9

where $w_1$ is the component of deflection derived from simply supported boundary conditions, while $w_2$ is the deflection produced by edge fixed moments. As established above, the bridge decks under investigation are orthotropic, having different flexural stiffness in the $x$ and $y$ directions. For this reason, it is necessary to modify the equation of equilibrium to include anisotropy. Huber (Timoshenko and Woinowski-Krieger (1959)) suggested the following treatment for reinforced concrete slabs:

$$D_x \frac{\partial^4 w}{\partial x^4} + 2(D_1 + 2D_{xy}) \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 y}{\partial y^4} = q$$  \hspace{1cm} 10

where $D_x$ and $D_y$ are the plate flexural stiffness in those respective directions. We can therefore accommodate anisotropy by introducing a new variable:

$$y_1 = y \sqrt{\frac{D_x}{D_y}}$$  \hspace{1cm} 11

As indicated earlier, bridge deck testing is conducted by applying load through a rectangular pad. For this reason and because of the formulation used in this model, the deck surface is then separated into two regions along the $y$-axis, namely within the loaded area, and outside the loading area. Boundary conditions for the interface between these regions are selected to create a single continuous deflected surface, satisfying the conditions of similar deflection, slope, moment and shear at the boundary between loaded and unloaded regions. Considering first the region inside the load pad, $w_1$ can be calculated from:
\[ w_1 = \frac{4qa^4}{D_x \pi^5} \sum_{m=1,3,5}^\infty \frac{(-1)^{(m-1)/2}}{m^2} \sin \frac{m\pi u}{2a} \{Y_1\} \sin \frac{m\pi x}{a} \]

where:

\[ Y_1 = 1 - \cosh \frac{m\pi y_1}{a} \left[ \cosh(\alpha_m - 2\gamma_m) + \gamma_m \sinh(\alpha_m - 2\gamma_m) + \alpha_m \frac{\sinh2\gamma_m}{2\cosh\alpha_m} \right] \]

\[ + \frac{\cosh(\alpha_m - 2\gamma_m)}{2\cosh\alpha_m} \frac{m\pi y_1}{a} \sinh \frac{m\pi y_1}{a} \]

in which, \( \alpha_m = m\pi b/2a \), \( \gamma_m = m\pi v/4a \).

In the region outside the loading pad, \( w_1 \) can be calculated from:

\[ w_1 = \frac{qa^3}{D_x \pi^4} \sum_{m=1}^\infty \frac{1}{m^4} \sin \frac{m\pi \xi}{a} \sin \frac{m\pi u}{2a} \{Y_2\} \sin \frac{m\pi x}{a} \]

where:

\[ Y_2 = \left[ \frac{2a}{m\pi} + y_1 - \frac{v}{2} \right] e^{-\frac{m\pi(2\gamma_1-v)}{2a}} - \left[ \frac{2a}{m\pi} + y_1 + \frac{v}{2} \right] e^{-\frac{m\pi(2\gamma_1+v)}{2a}} \]

The second component of the final deflection profile \( (w_2) \) accounts for a fixed condition which may be present at the edges \( x = (0, a) \). To account for this, the above equations are derived to determine slope at the support. Using the stiffness of the deck at that location, a moment can be applied along \( x = (0, a) \) to simulate a fixed condition. Such moments are described by:

\[ (M_x)_{x=0,a} = \sum_{m=1}^\infty E_m \sin \frac{m\pi y_1}{a} \]

where \( E_m \) is calculated such that the slope produced by moment \( M_x \) is exactly equal and opposite to the slope at \( x = (0, a) \). Following this method and for the finite loading pad area, \( E_m \) is found to be:

\[ E_m = \frac{2qa^2}{m^2 n^2 b \sin \frac{m\pi y_1}{b} [\tanh\alpha_n (\alpha_n \tanh\alpha_n - 1) - \alpha_n]} \]
Finally, $w_2$ is calculated using a standard formulation for a plate loaded by only moments along its edge:

$$w_2 = \frac{b^2}{2\pi^2 D} \sum_{m=1,3,5}^{\infty} \frac{\sin \left( \frac{m\pi (y_1 + b/2)}{b} \right)}{m^2 \cosh \alpha_m} E_m \left[ a_n \tan \alpha_n \cosh \frac{m\pi (x - a/2)}{b} \right.$$

$$\left. - \frac{m\pi (x - a/2)}{b} \sinh \frac{m\pi (x - a/2)}{b} \right]$$

Equation 9 can then be used to calculate the total deflection for slabs cast monolithically with girders. In experimental validation cases where slabs were simply supported, $w_2$ is omitted.

### 8.2.3.2 Finite width with edge supports

As $b/a$ ratio becomes smaller, the boundaries at $y=\pm b/2$ become more significant to the solution.

Again, beginning with a plate simply supported on all four sides, we will consider a general solution, following Timoshenko and Woinowski-Krieger (1959). For the loaded region of the plate:

$$w_1 = \sum_{m=1,3,5}^{\infty} \left\{ a_m + A_m \cosh \frac{m\pi y}{a} + B_m \frac{m\pi y}{a} \sinh \frac{m\pi y}{a} \right\} \frac{m\pi x}{a}$$

where

$$a_m = \frac{4qa^4}{\pi^5 m^5 D} (-1)^{(m-1)/2} \sin \frac{m\pi u}{2a}$$

For the region outside the loaded area:

$$w_1 = \sum_{m=1,3,5}^{\infty} \left\{ A_m \cosh \frac{m\pi y}{a} + B_m \frac{m\pi y}{a} \sinh \frac{m\pi y}{a} + C_m \sinh \frac{m\pi y}{a} \right\} \frac{m\pi x}{a} + D_m \frac{m\pi y}{a} \cosh \frac{m\pi y}{a}$$

where the constants $A_m, A'_m, B_m, B'_m, C_m$ and $D'_m$ are function of geometry, load and stiffness. They are established from the first four boundary conditions (i.e. similar deflection, slope, moment and shear at the boundary between loaded and unloaded regions) along with the two additional boundary conditions of zero deflection and zero curvature at the edges $y=\pm b/2$, and can
be found in Timoshenko and Woinowski-Krieger (1959) by solving six simultaneous equations. Deflection \( w_2 \) is determined using the same method as in the case of an infinite plate, by solving for \( E_m \) outside and inside the load pad, and applying resulting moments along the supports.

8.2.3.3 Finite plate with free edges

For practical reasons, many laboratory tests are conducted on bridge deck sections supported on only two opposing edges, with the remaining two edges free, which could produce different results, depending on the width \( b \), than ‘infinitely long’ actual in-place bridge decks (Nelson et al (2013a)). This case is similar to the finite width plate case above (i.e. Equations 19 and 21 can be used to calculate \( w_1 \) within and outside the loaded area, respectively), only with the following different boundary conditions (Timoshenko and Woinowski-Krieger (1959)):

\[
\left( \frac{\partial^2 w}{\partial y^2} + \nu \frac{\partial^2 w}{\partial x^2} \right)_{y=\pm b/2} = 0
\]

\[
D \left[ \frac{\partial^3 w}{\partial y^3} + (2 - \nu) \frac{\partial^3 w}{\partial x^2 \partial y} \right]_{y=\pm b/2} = 0
\]

The first four boundary conditions (i.e. similar deflection, slope, moment and shear at the boundary between loaded and unloaded regions) along with the two additional boundary conditions in Equations 22 and 23 result in six simultaneous equations for the constants \( A_m, A'_m, B_m, B'_m, C'_m \) and \( D'_m \). Four of which are identical to the previous case of edge supports and can be found in (Timoshenko and Woinowski-Krieger (1959)), while the remaining two equations were formulated as follows:

\[
A'_m \cosh \alpha_m (1 - \nu) + B'_m ((1 - \nu) \alpha_m \sinh \alpha_m + 2 \cosh \alpha_m) + C'_m \sinh \alpha_m (1 - \nu)
\]

\[
+ D'_m ((1 - \nu) \alpha_m \cosh \alpha_m + 2 \sinh \alpha_m) = 0
\]

\[
A'_m \sinh \alpha_m (3 - \nu) + B'_m \left( \alpha_m \cosh \alpha_m (3 - \nu) + \sinh \alpha_m (5 - \nu) \right) + C'_m \cosh \alpha_m (3 - \nu)
\]

\[
+ D'_m \left( \alpha_m \sinh \alpha_m (3 - \nu) + 2 \cosh \alpha_m (5 - \nu) \right) = 0
\]
Deflection $w_2$ is determined using the same method as in the case of an infinite plate, by solving for $E_m$ outside and inside the load pad, and applying resulting moments along the supports.

**8.2.3.4 Failure criterion**

The punching shear failure criterion adopted in this study was proposed by Muttoni (2003). It is based on the relationship of an effective shear stress at the punching cone perimeter to the shear strength capacity of concrete of a given strength. The effective punching shear stress was found to be strongly correlated with the state of cracking around the concentrated load, which was applied through a column in the case on Muttoni (2003), or applied through the loading pad in this case. A formulation to describe this cracking was developed based on the maximum rotation (in radians) at any point of the slab $\theta$ (available from the previous section) and its effective depth $d$.

Based on numerous test results, Muttoni (2003) proposed the following relationship for the effective punching shear stress at the punching perimeter:

$$\tau_r = \tau_c / (0.4 + 0.125 \theta d k_D)$$

The constant $\tau_c$ represents the shear strength of concrete and $k_D$ accounts for the maximum aggregate size of the concrete (proposed by Vecchio and Collins (1986)), as follows:

$$\tau_c = 0.3\sqrt{f'_c}$$

$$k_D = 48/(D_{max} + 16) \geq 1.0$$

The punching shear load $P_r$ can then be calculated using the effective punching stress and the transverse area of the critical perimeter $b_0$, as follows:

$$P_r = \tau_r b_0 d$$

The perimeter $b_0$ extends $d/2$ from the edge of the concentrated load (AASHTO LRFD).

**8.2.4 Analytical procedure**
Based on the equations and moment-curvature responses developed above, the process described here develops a full load-deflection field response up to failure, which could be flexural or likely for this type of deck slabs punching shear. The procedure is coded using MATLAB program based on the following algorithm:

**Step 1:** The slab under investigation is effectively discretized into nodes in the x and y direction. A 10 by 10 mm grid size was used in the present study. Each node corresponds to an (x,y) coordinate on the slab surface.

**Step 2:** The complete moment-curvature responses of the slab, for a unit width, are established in x and y directions up to flexural failure, for positive and negative bending, using cracked section analysis (a sample is shown in Figure 8-3). This process accounts for concrete non-linearity and cracking as well as composite or non-composite action between concrete and FRP SIP form.

**Step 3:** An increment of loading pressure $q$ is applied to the loaded area of the slab. Depending on the boundary conditions of the slab, the relevant plate theory equations are executed over the entire area of the plate, or over a smaller user selected area of interest. Moment restraint is applied at the edges in varying intensity depending on input from the user. The flexural stiffness in x and y directions are assumed at this first step. The result is a deflection field based on the load $q$.

**Step 4:** Based on the deflection field from step 3, slope and curvature are calculated using a finite difference algorithm in the x and y direction, or directly from derivatives of the above formulas. Using the moment-curvature plots established in step 2, the flexural stiffness in both directions can then be established at these values of curvatures (a sample is shown for specimen FA1 (Table 1) in Figure 8-5). Stiffness profiles along the spans indicate the severity of cracking at any particular location.

**Step 5:** Using the new stiffness values established in step 4, a new complete iteration of the program is carried out by repeating step 3. The process is repeated until the stiffness of the plate
in each direction has converged. At the end, the correct deflection field of the slab under the loading $q$ is established, including the mid-span maximum deflection $w$. The values $(q,w)$ represent one point on the predicted load-deflection response.

**Step 6:** Check the shear failure criterion and establish the corresponding pressure that causes shear failure $q_{ult-shear}$. If the applied $q$ in step 3 is larger than or equal to $q_{ult-shear}$, then failure has occurred and analysis stops. If not, proceed to step 7.

**Step 7:** The pressure $q$ is increased by a given increment and steps 3-6 are repeated until the full load-deflection response of the deck is developed. If shear failure checked in step 6 does not govern, the process continues until the moment-curvature response indicates a flexural failure at load $q_{ult-flexural}$. This may only occur in slabs with very long spans, but for the bridge deck slabs studied here, punching shear typically governs.

### 8.3 Model evaluation

Several experimental studies are used to evaluate the model, using 29 different scaled and full scale tests. A summary of these experimental tests and their references is provided in Table 1. The tests cover a very wide variety of deck slabs including flat-ribbed or corrugated GFRP forms, decks with bonded or unbonded GFRP forms, decks with monolithic connections to the supporting girders or simply supported on rollers, deck sections of finite widths of various aspect ratios to span length and tests carried on one large deck (i.e. `infinite width`) of a complete bridge system with all details including multiple support girders and continuity in both directions.

In all tests, the primary failure mode was punching shear of concrete, which was also consistently predicted by the model as shown in Figure 8-6 and demonstrated in detail in the following sections. Table 1 also provides a summary of the experimental and predicted ultimate loads along with the percent difference, where negative and positive values indicates an under prediction (conservative) and over prediction, respectively. The average percent difference is -5.5%, which shows that overall, the model is slightly conservative over the given population of test results.
The standard deviation of 13.6 is fairly high, but not surprising given the large variety of detailing and slab geometries within the population.

### 8.3.1 Bond effect of flat ribbed forms in finite width and infinite width small scale decks

Figure 8-7 shows the measured and predicted load-deflection responses for: (a) composite slabs A2 and FA1 (Table 1) with adhesive bond between concrete and GFRP forms, and (b) unbonded slabs B2 and FA5 (Table 1) with no adhesive or mechanical bond at the interface. In all specimens flat GFRP forms with T-shape ribs were used and the specimens were small scale of 1:2.75. Specimens A2 and B2 were sections of finite width of 0.9 the span, while FA1 and FA5 were identical in configuration and span as A2 and B2 but were part of a complete bridge with long girders of more than 4.5 times the deck span (referred to here as infinite-width). Figure 8-7 compares model predictions to experimental results for the load-deflection responses and the maximum load with the appropriate failure mode, namely punching shear. Also shown for reference, the hypothetical flexural failure load, which is consistently higher than punching shear, and hence, does not govern. The model, however, does not capture accurately the non-linear response near maximum load.

It can be concluded from Figure 8-7 that the lack of bond at FRP-concrete interface results in noticeable reduction in stiffness but little reduction in ultimate load. Also, the finite-width specimens, which are much more practical in experimental research, could lead to lower stiffness but little reduction in ultimate load, hence are considered conservative.

### 8.3.2 Bond effect of corrugated forms in finite width full scale decks

Figure 8-8 shows the load-deflection responses of full scale deck slabs S2 and S3 with corrugated GFRP forms, where S2 was adhesively bonded while S3 was not. Also shown for comparison is adhesively bonded full scale D2 with flat-ribbed GFRP form. Figure 8-8 shows reasonable prediction of punching failure loads though stiffness was underestimated. The flat-ribbed form deck D2 failed first in punching shear, but the load in this specimen continued to increase slightly
at a very shallow slope until the splice of the GFRP forms failed. It should be noted that the GFRP flat-ribbed sections were significantly thicker than the corrugated forms. Also, similar to the previous section, lack of adhesive bond in the corrugated form decks resulted in significantly lower stiffness but little reduction in failure load. It is apparent that the model underpredicts the flexural stiffness of the corrugated system, to a greater extent than the T-up system. This may be because the model does not accurately capture the discrete nature of the crests and valleys of the corrugation. The model uses smeared properties for the traffic direction, in this case using the shallowest profile for the flexural stiffness. In reality, during the test load is shed directly through the two closest valleys (forming stiff ribs in the slab) and the longitudinal stiffness is less consequential.

8.3.3 Width effect in flat ribbed forms of small scale decks
Figure 8-9(a) shows the load-deflection responses of finite-width specimens A1, A2 and A3 of widths equal to 0.43, 0.9 and 1.33 the span; along with `infinite-width` specimen FA3 of width equal to 4.5 the span. The model predicts the peak loads reasonably well but for narrow decks (A1), it underestimates stiffness. Again, the model cannot capture the flat plateaus around the peak load, which are associated with the secondary GFRP progressive splice failure that follows punching shear. For practical purposes, finite-width deck specimens of 1.3 the span seem to represent the actual full bridge condition reasonably well.

Figure 8-9(b) shows the deflected shape of the decks along the center line in direction parallel to traffic, at loads of 60 and 90 kN, except for narrow deck A1 which did not reach 90 kN. The model generally showed similar trends to experiments, where the deflection reduces further away from the loading point.

8.3.4 Deck span effect in flat ribbed forms of small scale specimens
Figure 8-10 shows the load-deflection responses of tests FA3, FB3 and FC3, all part of one full bridge specimen including complete arrangements of four girders with end diaphragms. FA3 and
FB3 are exterior and interior equal spans (10.2 times the deck thickness), respectively, while FC3 is an exterior span that is 33% longer than FA3 (i.e. 13.6 times the deck thickness). Figure 8-10 shows that the model predicted reasonably well the overall behavior but slightly underestimated the punching shear peak load. It can be seen that the interior span FB3 shows higher stiffness and strength due to the restraints arising from continuity, compared to exterior span FA3. Also, the longer span FC3 is lower in stiffness than FA3 as would be expected, but its ultimate load is only slightly lower than FA3.

8.3.5 Assessment of punching shear failure criteria

The punching shear failure criterion (Muttoni (2003)) used in this model (Equation 35) is assessed in Figure 8-11 by plotting \( \frac{\tau_r}{\tau_c} \) from the model and from the experimental data versus \( \theta d k_D \) for all the cases tested and given in Table 1. Four additional test results correspond to conventional steel reinforced concrete control specimens tested and reported in Nelson and Fam (2013a and 2013b) are also shown in Figure 8-11 and denoted with hollow markers. Generally, the full scale decks with flat-ribbed forms were predicted best by the failure criterion, while the full scale decks with corrugated plates were slightly under predicted. Nearly all scaled deck results were predicted conservatively. A general trend was observed whereby better restrained deck panels (e.g. interior spans of full bridge) plotted further from the failure envelope. For this reason, the failure envelope should be considered as a confident predictor of capacity in relatively unrestrained decks, while better restrained deck pads will be predicted conservatively. It should be noted that the latter is the norm in practical bridge applications. The significant conservatism of the punching shear failure criteria in some scaled deck slabs could be attributed to scale effect, especially with regard to shear strength of concrete which is quite sensitive to section depth.

A careful examination of the above evaluations shows that while the model is able to generally capture the effect of various parameters on the performance of the decks in question, it
does not always produce a perfect prediction. Divergence between model and test results can occur for several reasons. Natural variability in the performance of test slabs could explain some divergence, along with assumptions which were explicitly made above in establishing the model. Additional sources of difference may arise from localized phenomena which are not captured by this model such as horizontal shear failure within the FRP form panels, or damage at the panel to panel splice location.

8.4 Parametric investigation

Using the established model, a parametric study was carried out to expand existing test results through a range of practical applications. There are certain configurations of FRP SIP formwork decks which are either impossible or impractical to test in the laboratory, including full scale decks cast integrally in a full bridge. Some additional parameters such as FRP reinforcement ratio are limited to theoretical investigations due to the limited availability of commercial (off-the-shelf) FRP panel products. The following sections examine a variety of parameters not investigated in previous laboratory work.

8.4.1 Effect of FRP reinforcement ratio (SIP form thickness) in full scale decks

The experimental studies utilized readily available off-the-shelf GFRP sections as SIP forms for the concrete decks, which provided only limited selection for plate thickness. In this section, the effect of plate thickness is varied to understand its impact on the deck performance. A full scale deck, identical to D2 (Table 1) is considered in this case, except that the GFRP plate thickness is varied from 4 mm to 16 mm, representing FRP reinforcement ratio of 2.7% to 10.7%, while maintaining the same laminate structure of the GFRP plate as in D2. For comparison, the minimum steel reinforcement ratio, which is the design value, as per the Canadian Highway Bridge Design Code (CSA S6-06) of 0.003 transforms to a 2.25% reinforcement ratio, which is close to the low end of this parametric investigation. Figure 8-12 shows the load-deflection responses as well as the design service load based on half-axle load of HL-93 truck plus dynamic
allowance (AASHTO (2007)). It can be seen that the reinforcement ratio affects flexural stiffness after cracking and also the punching shear failure load, which reduced by about 20% as reinforcement ratio reduced from 10.7% to 2.7%. It should be noted that while punching shear is a concrete-related failure criterion, Equation 35 shows that punching shear strength is a function of the maximum rotation of slab, which in turn is affected by flexural stiffness (i.e. by FRP reinforcement ratio).

8.4.2 Effect of aspect ratio of full scale decks

For practical reasons, the critical width effect involved in testing bridge deck sections in the laboratory has never been investigated experimentally at full scale. On the other hand, the model has been evaluated against this parameter using 1:2.75 scale testing (Figure 8-9). In this section, a case identical to D2 (Table 1) is considered, except that deck widths of 1.0, 1.6, 2.4 and 5.0 m are modeled, representing (width/deck span) aspect ratios of 0.55 to 2.73. Figure 8-13 shows the resulting load-deflection curves. The 5 m wide deck closely matched the results for an infinitely long deck. Generally, wider deck specimens are stiffer and stronger due to the additional distribution of load, reaching a maximum when no additional load can be shared longitudinally. The ultimate loads obtained for aspects ratios of 2.73, 1.33, 0.87 and 0.55, were 100%, 94%, 83%, and 73%, respectively, of the ultimate load of the real condition of infinite width. Based on the results shown in Figure 8-13, experimental researchers can form an idea on how their limited-width specimens behave relative to the real case, and perhaps can even estimate the strength and stiffness of the hypothetical real case using extrapolation of their test data from the limited-width specimens.

8.4.3 Effect of span length (girder spacing) in full scale decks

The effect of girder-to-girder spacing (i.e. span of the deck) was studied at small scale using only two cases, which were used in the model validation (Figure 8-10). In this section, decks identical to D2 (Table 1), except that the deck span is varied from 1.83 m to 3.05 m, are analyzed for two
different GFRP plate thicknesses, namely 13 mm (Figure 8-14(a)) and 4 mm (Figure 8-14(b)). The analysis for each span is carried out for two cases; a case of fixed deck width of 1.83 m (i.e. variable aspect ratio), and a case of constant aspect ratio (i.e. variable width). Figure 8-14 shows that as the span is increased, the results of the two series diverge increasingly to the point where they are no longer representative of one another. For this reason, care should be taken to select the proper aspect ratio when testing bridge deck sections. A second interesting observation is that decks failed in punching shear even at large, but practical, spans with low FRP reinforcement ratio. As indicated earlier, the punching shear failure criterion depends on rotation. As such, the punching shear load is reduced in cases of lower flexural stiffness. For this reason, punching shear still occurs before flexural failure even in relatively long and low-FRP reinforced slabs but at lower load (for example, the ultimate load reduces by about 18% as the deck span-to-depth ratio increases from 10 to 16.5). It should be noted that this only holds for the case of a single concentrated load. As spans become longer in a real bridge, the live load distribution may allow multiple loads to accumulate in one span which could promote flexural failure over punching shear.

8.5 Summary and conclusions

A robust analytical model for predicting full response and ultimate load of concrete bridge decks constructed with fiber-reinforced polymer (FRP) stay-in-place (SIP) structural forms is presented. It adopts the plate theory to establish surface deflections, while incorporating concrete nonlinearity in compression and cracking in tension, as well as the degree of bond between the FRP SIP form and concrete. The model accounts for various boundary conditions at the edges of the deck in both directions, including both finite and infinite width in the direction of traffic, and either fixed or hinged conditions in the other direction, depending on the connection to support girders. A punching shear failure criterion was incorporated to predict the ultimate load. The model was evaluated against a large experimental database and reasonable agreement was
observed. The average percent difference in ultimate loads was 5.5%. The model was then used in a parametric study to assess FRP reinforcement ratio, in terms of the FRP plate thickness, width of the deck parallel to traffic, and span of the deck, which is the girder spacing. It was shown that reducing FRP reinforcement ratio from 10.7% to 2.7% results in about 20% reduction in punching shear ultimate load. The ultimate loads obtained for decks with (width/span) aspect ratios of 2.73, 1.33, 0.87 and 0.55, were 100%, 94%, 83%, and 73%, respectively, of the ultimate load of the real condition of infinite width. Finally, the punching shear load reduced by about 18% as the deck span-to-depth ratio increased from 10 to 16.5. Although mainly developed for calculating bridge deck capacity, the model is also sufficient in predicting the flexural stiffness of FRP SIP decks. The flexural stiffness of model predictions versus experimental results for tests displayed in Figure 8-4 resulted in an average 4% difference with a 13% standard deviation, demonstrating a good correlation.

8.6 References


Muttoni, A. (2003). “Shear and punching strength of slabs without shear reinforcement”, Beton-und Stahlbetonbau, 98(2), 74-84. (in German)


Table 8-1: Test matrix used to evaluate the model

<table>
<thead>
<tr>
<th>Specimen</th>
<th>scale</th>
<th>Width (m)</th>
<th>Span (m)</th>
<th>$P_{\text{test}}$ (kN)</th>
<th>$P_{\text{model}}$ (kN)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>D2</td>
<td>1:1</td>
<td>1.60</td>
<td>1.83</td>
<td>691.2</td>
<td>752.2</td>
<td>8.8</td>
</tr>
<tr>
<td>D3</td>
<td>1:1</td>
<td>1.60</td>
<td>1.83</td>
<td>748.0</td>
<td>702.5</td>
<td>-6.1</td>
</tr>
<tr>
<td>S2</td>
<td>1:1</td>
<td>1.60</td>
<td>1.83</td>
<td>588.7</td>
<td>561.0</td>
<td>-4.7</td>
</tr>
<tr>
<td>S3</td>
<td>1:1</td>
<td>1.60</td>
<td>1.83</td>
<td>528.9</td>
<td>453.9</td>
<td>-14.2</td>
</tr>
<tr>
<td>S4</td>
<td>1:1</td>
<td>1.60</td>
<td>1.83</td>
<td>427.0</td>
<td>438.6</td>
<td>2.7</td>
</tr>
<tr>
<td>A1</td>
<td>1:2.75</td>
<td>0.286</td>
<td>0.665</td>
<td>86.0</td>
<td>72.2</td>
<td>-16.0</td>
</tr>
<tr>
<td>A2</td>
<td>1:2.75</td>
<td>0.604</td>
<td>0.665</td>
<td>120.0</td>
<td>111.7</td>
<td>-6.9</td>
</tr>
<tr>
<td>A3</td>
<td>1:2.75</td>
<td>0.886</td>
<td>0.665</td>
<td>141.0</td>
<td>121.4</td>
<td>-13.9</td>
</tr>
<tr>
<td>B1</td>
<td>1:2.75</td>
<td>0.286</td>
<td>0.665</td>
<td>63.0</td>
<td>61.9</td>
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| Average (%) | -5.5 |
| Standard deviation (%) | 13.6 |

Figure 8-1: (a) Schematic of typical slab-on-girder bridge deck with FRP SIP structural form, (b) photo of bridge decks tested with FRP SIP forms

Figure 8-2: Geometry and stresses and strain profiles used in cracked section analysis of fully and partially composite sections
Figure 8-3: Sample moment-curvature response for deck specimen FA1 (Table 1)

Figure 8-4: Experimental versus analytical stiffness of decks based on slope of load-deflection curve
Figure 8-5: Sample inertia-curvature profile for deck specimen FA1 (Table 1)

Figure 8-6: Experimental versus analytical punching shear failure loads
Figure 8-7: Experimental versus analytical load-deflection responses for (a) bonded slabs FA1 (dashed) and A2 (solid), and (b) unbonded slabs FA5 (dotted) and B2 (dashed).

Figure 8-8: Model and test results for bonded slabs D2 with flat-ribbed form and S2 with corrugated form and unbonded S3 with corrugated form.
Figure 8-9: (a) Experimental versus analytical load-deflection responses for slabs A1, A2, A3 and FA3 of different widths and (b) centerline deflection profiles (parallel to traffic) for the same slabs at various load levels.

Figure 8-10: Experimental versus analytical load-deflection responses for exterior slabs FA3
Figure 8-11: Comparison of Muttoni (2003) punching shear failure criterion with test data

Figure 8-12: Effect of panel thickness on performance of FRP SIP full scale decks
Figure 8-13: Effect of full scale deck section width on performance

Figure 8-14: Effect of span length on full scale decks with (a) constant width and (b) constant aspect ratio
Chapter 9

Summary and conclusions

9.1 Introduction
Stay-in-place formwork using FRP composite panels presents a very promising avenue for improving the constructability and durability of infrastructure, particularly bridge decks. This thesis describes a significant study which has examined a number of features of the FRP SIP formwork system. Conclusions from the study are listed below, organized by chapter.

9.2 GFRP permanent forms with T-shaped ribs for bridge decks
There are, in principle, no technical barriers to applying FRP SIP form technology on a much wider scale. With time and future research, the technology is expected to reach a greater share of the bridge deck market and be specified alongside conventional reinforced concrete products.

In this study, a commercially available GFRP panel was used as a novel stay-in-place (SIP) formwork for concrete bridge decks. The panel also replaces the bottom layer of reinforcement. The panel combined with a face GFRP plate was also tested as an ‘all-GFRP’ deck as currently being used commercially. Full scale testing was conducted using a system designed to replicate field conditions at the connections between the deck and standard AASHTO precast concrete girders. A conventional control RC deck was designed for the same span and tested. Auxiliary specimens were also tested to isolate the performance of the critical GFRP panel-to-panel splice region. From the results of this study, the following conclusions can be drawn:

1. The optimal concrete deck system, with GFRP SIP form and FRP rebar, showed remarkable ultimate strength and pseudo-ductility. A peak load, 7.8 times higher than HS-25 service load plus impact was achieved. It also met the most stringent US DOT requirements for deflection limits of L/1600 at service.
2. Punching shear failure in the RC deck is catastrophic in that the full strength is lost suddenly. Punching shear also occurs in the GFRP SIP form deck, at the same load as the RC deck, however, it is not catastrophic. Stiffness reduces, but the deck continues to gain strength, until the mechanical fasteners of the lap-splice fail by tilting and tearing through GFRP. The load drops gradually in a step fashion as the fasteners fail progressively.

3. Adhesive bond failure of the lap-splice starts within a load range of about 2.8 to 3.6 times the maximum service load and approximately 58% of the load causing punching shear failure. It results in slight reduction in stiffness but the deck continues to gain strength.

4. The absence of the top layer of GFRP rebar in the GFRP SIP form deck eliminated the reserve strength of about 17% beyond the punching shear failure and led to a wider crack parallel to the girders in the negative moment region.

5. Testing bridge decks under simply supported conditions results in a significant reduction in stiffness at service load, relative to a deck with the proper monolithic connection to girders. In the GFRP SIP form deck, deflection was larger by about three times using simply supported boundary conditions, however, ultimate strength was lower by only 13%, and hence, the approach of simply supported specimens is conservative.

6. The peak strength of the ‘all-GFRP’ deck tested in this study was 3.7 times higher than HS-25 service load plus impact, however, deflection at service exceeded the L/800 limit. While the ultimate strength was 37% lower than the RC deck, the deck showed remarkable pseudo-ductility. It should be noted that the thickness of the deck was 26% smaller than other specimens.

9.3 A new deck system using corrugated FRP panels

The behaviour of a new GFRP SIP structural form system, composed of corrugated panels with pin-and-eye interlocking joints, for concrete bridge decks has been studied. Full scale specimens were constructed and tested to failure under a single load. Much attention was given to simulate
the construction details of the monolithic connection between the deck and the supporting AASHTO type III precast girders. This included the typical rough finish of the top surface of the girder and the protruding stirrups. A specimen was built and tested to examine the feasibility of using GFRP SIP forms as an overhang beyond the fascia girder. The study examined the effect of adhesive bond at the interface between concrete and the form as well as the thickness of the form on performance. The behaviour was also compared to a control specimen reinforced by to top and bottom orthogonal mesh of steel rebar. The following conclusions can be drawn:

1. The ultimate strength of all interior span decks with GFRP SIP forms ranged from 3.5 to 4.9 times the half-axle service load plus impact of the CL-625 and HS-25 design trucks of the CHBDC and AASHTO design specifications, respectively.

2. The ultimate strength of all interior span decks with GFRP SIP forms ranged from 0.62 to 0.86 the strength of the control RC deck. Although the over thickness of both decks was identical, it should be noted that the average thickness of the SIP-form deck was only 0.78 that of the RC deck because of the corrugations.

3. The deflections of all interior span decks with GFRP SIP forms at service loads of the CL-625 and HS-25 design trucks were well below the span/1600 limit. Also, maximum strains in the GFRP forms at service were less than 2 percent of ultimate values.

4. Failure of all interior span decks with GFRP SIP forms was by punching shear, similar to the RC control deck. This punching shear and associated debonding initiated a longitudinal tearing at the corner between the bottom flange and web at supports, developing into an inclined fracture of the web.

5. No failure or damage was observed at any of the pin-and-eye transverse connections.

6. The post-peak load response of all interior span decks with GFRP SIP forms was superior to that of RC control deck, in that the load dropped very gradually over a very large range of
deflection, due to the progressive tearing of the GFRP form. On the other hand, once punching shear failure occurred in the RC deck, the load dropped significantly and suddenly.

7. No horizontal slip was observed between GFRP SIP forms and supporting girders. This suggests that construction procedure and detailing used to ensure anchorage and monolithic connection worked well. This includes seating the SIP form over haunches on the inner edges of girders, grouting underneath the 90 mm embedment of the form, and then casting the deck.

8. Using wet adhesive bond at the interface of freshly cast concrete and the GFRP form resulted in significant enhancement of stiffness but very little gain in ultimate strength.

9. Reducing the GFRP SIP form thickness by 24 percent reduced the ultimate strength of the interior span deck by 20 percent.

10. GFRP SIP forms can be used in overhangs at exterior girders, provided that a proper splicing method is used to connect the overhang form to the interior span form. In this study, narrow GFRP straps with self-taping screws were used successfully to connect the forms. The 1070 mm long overhang deflected 16 mm under the weight of concrete. It is suggested to position the overhang form slightly inclined upwards to compensate for this kind of deflection.

While this study has demonstrated very positive performance of this novel deck under monotonic loading and indicated a strong potential of the system, future studies should address the performance of the system under cyclic fatigue loading and freeze-thaw cycles. These two effects are particularly important with regard to the interface bond conditions between the GFRP forms and concrete. Given the promising response of the system observed in this study, the authors have started planning for the long-term studies of the system.

9.4 Full bridge deck testing at scale with FRP SIP formwork

The following conclusions can be drawn from this experimental study:
1. The structural response of the FRP SIP form system is generally similar to conventional RC bridge decks. Performance is initially dominated by flexure while ultimate capacity is governed by punching shear and is influenced by the amount of available in-plane restraint.

2. The location of loading has a profound impact on the capacity. Pads in the interior span near the center of the deck achieved 18% higher loads due to the increased in-plane restraint, compared to pads in exterior spans. Stiffness was also increased by up to 65%.

3. Slab edge locations present the weakest restraint and correspondingly the lowest deck capacities, 24% lower than mid-span.

4. When loading is applied directly above the panel-to-panel splice, providing no mechanical or adhesive bond at splice has no impact on ultimate strength. If loading is applied half-way between splices, providing bond resulted in a 21% increase in capacity.

5. Providing full composite action at the FRP-concrete interface using adhesive bond increased ultimate strength by 29%.

6. The two different deck spans tested, which represent common girder spacing of 1830 and 2440 mm, failed by punching shear at the same ultimate load, however, flexural stiffness was reduced in the longer span.

7. The slight reduction in stiffness and ultimate strength in the FRP SIP form system, compared to the RC system, can be attributed to the 15% smaller thickness of the FRP SIP form deck, due to the haunch over the girder. Peak load capacities for the FRP SIP form system were between 2.8 and 4.3 times the ultimate factored live load.

Further research is underway at Queen’s University to optimize the detailing of the panel-to-panel splice for improved continuity. Fatigue and durability testing are also underway to address possible concerns involving water ingress and frost jacking of the FRP panels.

9.5 The effects of splice and bond on performance of FRP SIP decks

Based on this experimental study, the following conclusions can be made:
1. Width-to-span (w/s) aspect ratio of concrete bridge deck sections with FRP stay-in-place (SIP) forms has a profound impact on ultimate strength, stiffness and failure mode. The minimum (w/s) ratio that should be used to simulate real conditions in full bridges is 1.5.

2. At (w/s) ratios below 0.6, one-way shear failure occurs at a much lower strength than that of full bridges. Between (w/s) of 0.6 and 1.5 punching shear failure occurs at increasing load levels but perimeter of failure surface may intercept free edges depending on crack angle, thereby affecting full resistance. Additionally, increasing in-plane restraint is available in wider decks. Beyond (w/s) of 1.5, ultimate strength is constant as punching surface is fully contained within the deck width and full amount of restraint is available.

3. Increasing concrete compressive strength from 17 to 42 MPa increased deck ultimate capacity by about 20% but did not influence stiffness. In both cases punching shear failure occurred.

4. The ultimate peak load is not necessarily the punching shear load, but could be higher and occurring at larger deflection, particularly when low strength concrete is used. This is because of the considerable shear reinforcement provided by FRP ribs of the form, which enhance ultimate shear strength significantly beyond punching shear load.

5. Applying adhesive bond at the FRP SIP-concrete interface increased the deck ultimate strength and initial stiffness by 30% and 73%, respectively, due to composite action.

6. In decks with adhesive bond between concrete and FRP form, loading directly above FRP splice resulted in a 20% lower strength than loading half-way between splices. This is an opposite trend to that observed in decks without adhesive bond and reported in literature.

7. Monolithic connection between deck and support girders has little effect on ultimate strength governed by punching shear but measurable effect on stiffness, in comparison to decks simply supported on rollers (for convenience) during testing.
9.6 Splice optimization of FRP stay-in-place structural forms

Based on the results of the experimental study described in this article, several conclusions can be made regarding the splice detailing of FRP SIP formwork deck systems:

1. Using narrow FRP panels of small width-to-deck thickness (b/t) ratio produces significantly lower deck capacity when no panel-to-panel bond of any sort is provided at the overlap splice region. The deck with b/t=1.9 exhibited a 24% lower capacity than that with b/t=7.2.

2. Narrow FRP panels with unbonded lap splice may force punching shear crack to terminate at the splice, at a steeper angle, thereby producing lower punching shear capacity.

3. While FRP panel width has a significant effect on strength and stiffness in panels with unbonded lap splices, it has an insignificant effect on panels with bonded splices.

4. Deck specimens showed ultimate failure loads ranging from 5.7 to 8.8 times the equivalent service load at scale, depending on FRP panel width and splice bond.

5. All decks failed primarily in punching shear. In bonded lap splices, this was combined with lap splice delamination within the GFRP plate, not within the adhesive layer, and in unbonded overlaps, the splice simply slipped.

6. Having a layer of adhesive bond between concrete and FRP panels increased stiffness and ultimate strength by 27% in bonded splice decks.

7. For a GFRP plate overlap splice of a length equal to 4.3 times plate thickness, the optimal splice configuration was combined adhesive bond and mechanical fasteners of diameter equal to FRP plate thickness, spaced at 1.8 times the overlap length. This configuration developed 68% of the plate tensile strength, which can be increased by increasing overlap length.

8. Adhesive bond alone, at overlap splice of 4.3 times plate thickness, can also achieve 68% of plate tensile strength, whereas mechanical fasteners alone can only reach a maximum strength of about 14-45%, depending on diameter and spacing of fasteners.

9. While complete lack of bond (adhesive or mechanical) at plate overlap means a zero strength
in a splice coupon configuration, it is not the case when the same splice is placed within actual bridge deck configuration, where significant punching shear capacity can still be obtained without any splice bond.

9.7 Modeling of punching shear of FRP SIP formwork concrete bridge decks

This study presented a model for predicting the full response and ultimate load capacity of concrete bridge decks constructed with FRP SIP formwork. The model uses the theory of plates and shells, and accounts for concrete cracking and nonlinearity as well as the bond level between the FRP form and concrete interface. The model establishes the deformed shape and deflections of the deck surface under a given load representing half-axle load of a truck, applied over a specific rectangular foot-print of the tires.

The model accounts for various boundary conditions of the edges of the deck in both directions. In the direction of traffic, the model accounts for several end conditions, namely infinite length as in a full bridge, finite length with free edges as in common practice of experimental test specimens or finite length with end support as in deck ends near diaphragms. In the direction normal to traffic, the model accounts for fixed-end decks monolithically cast with the support girders, or simply supported edges as in some of the deck specimens supported on rollers during tests as in many experimental programs. The plate theory incorporating all the features mentioned leads to developing a full load-deflection response of the deck.

The model incorporates a punching shear failure criterion to predict the ultimate load, but also checks for flexural failure, which for common deck spans (girders spacing) does not govern. The punching shear failure criterion indirectly accounts for the flexural stiffness of the deck, which depends on the FRP section used and deck span. The failure criterion terminates the load-deflection response established through the plate theory at the failure load.

The model was evaluated against a large experimental database on about 30 bridge decks with FRP SIP forms of various configurations, boundary conditions, and sizes. Reasonable
agreement was observed and the average percent difference between predicted and experimental ultimate load was -5.5% (i.e. a conservative under prediction) with a standard deviation of 13.6%.

The model was then used in a parametric study to assess various parameters beyond the limitations of the experimental programs, namely the FRP reinforcement ratio, in terms of the plate thickness of the SIP form, width of the deck, which is the dimension in the direction parallel to traffic, and span of the deck, which is the girder spacing. It was shown that reducing the FRP reinforcement ratio from 10.7% to 2.7% results in about 20% reduction in the punching shear ultimate load. The ultimate loads obtained for decks with (width/ span) aspect ratios of 2.73, 1.33, 0.87 and 0.55, were 100%, 94%, 83%, and 73%, respectively, of the ultimate load of the real condition of infinite width. The punching shear ultimate load reduces by about 18% as the deck span-to-depth ratio increases from 10 to 16.5.

9.8 Recommendations for future work

Considerable research has been conducted to explore the concept of FRP stay in place (SIP) structural form, using various systems. Conclusions from research efforts have been very promising and a number of field applications have successfully demonstrated the feasibility of the system. There are nevertheless several areas requiring future work in order to achieve more widespread application of the system. These areas are:

1. Efforts need to be made to synthesize current design methods in order to produce concise design guidelines for FRP SIP structural form systems. Such a guide should encompass all the commercially available systems. The development of simplified analysis procedures to efficiently design SIP systems without the need to resort to detailed FE analyses is recommended.

2. Additional assessments of the in-place bridges built using the FRP SIP systems should be carried out to establish the actual durability of the system. Some field projects have been in
place for at least 10 years and such an assessment would be an excellent indication of the system’s longevity.

3. Work should still be conducted on the structural response of bridges built using FRP SIP structural form technology, at ultimate limit states. Thus far, global structural response has only been assessed at service loading in field applications.

4. A comprehensive economic life-cycle cost analysis of the technology with reference to field projects could improve the case for contractors and owners to specify FRP SIP form systems for infrastructure projects.

5. Concerns have been raised related to the ingress of moisture and possible frost jacking between FRP the form and concrete components of SIP form decks. While this issue has not been encountered in field applications, it is a problem common to steel SIP formwork systems and similar mitigation tactics could be employed.

6. Fire resistance is a concern for any system using exposed FRPs, but this has not been a barrier to the completion of field projects in the past and is largely at the discretion of the local department of transportation.