Pultruded GFRP Sections as Stay-in-Place Structural Open Formwork for Concrete Slabs and Girders

By

Hart Noah Honickman

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Queen's University
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ABSTRACT

Commercially available glass fiber-reinforced polymer (GFRP) off-the-shelf structural shapes have great potential as stay-in-place open structural forms for concrete structures, including bridge decks and girders. The system simplifies and accelerates construction, and the non-corrosive GFRP forms can fully or partially replace steel rebar. In this study, eight concrete slabs were constructed using flat pultruded GFRP plates, and nine girders were constructed using trapezoidal pultruded GFRP sheet pile sections as stay-in-place structural forms. No tension steel reinforcement was used. All specimens were tested in four-point monotonic uniaxial bending. Four adhesive and mechanical bond mechanisms were explored to accomplish composite action. The most effective mechanism, considering structural performance and ease of fabrication, was wet adhesive bonding of fresh concrete to GFRP. Although failure was by debonding, no slip was observed prior to failure. Other parameters studied were concrete slabs’ thicknesses and their shear span-to-depth ratios. For the girders, three different cross-sectional configurations were examined, namely, totally filled sheet piles, one with a voided concrete fill, and an all-GFRP box girder developed by bonding flat GFRP sheets to the upper flanges of the sheet piles with a cast-in-place concrete flange. Girders were tested in positive and negative bending to simulate continuity. The built-up box girders showed superior performance, with up to 70% higher strength and 65% lower weight than the totally filled sections. It was found that similar size conventional steel-reinforced concrete sections of comparable stiffness have considerably lower strength, while those of comparable strength have considerably higher stiffness than FRP-concrete members. An analytical model was developed to predict the behaviour and failure loads of slabs and girders, using cracked section analysis. A unique feature of the model is a multi-stepped failure criteria check that can detect flexural, shear, or bond failure. The model was successfully validated using the experimental results, and used in a parametric study. It was shown that using the typical value of 1MPa for shear strength of cement mortar predicts debonding failure, which occurs slightly above the interface, quite well. Also, in practical applications of longer spans, flexural failure is likely to occur prior to bond failure. The effects of concrete strength, thickness and configurations of the GFRP sheet pile, and varying the shear strength of the cement paste were also explored.
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NOTATIONS

$B$  Total width of section

$B_i$  Total width of the section at the elevation or layer number $(i)$ of interest

$B_{lf}$  Width of lower flange of GFRP sheet pile section

$B_{wt}$  Distance between outer surfaces of webs of GFRP sheet pile section at the location where the webs meet the lower surface of the upper flanges

$c$  Depth to neutral axis measured from extreme compression fibre

$d$  Depth of slab specimens measured from extreme compression fibre to centroid of flat GFRP plate

$d_{TS}$  Thickness of region within which tension stiffening is effective

$d_x$  Length of discrete region of interest along the length of the beam

$E_c$  Secant elastic modulus of concrete taken between a stress of 0 and $0.4 f_{c'}$

$E_{ci}$  Secant elastic modulus of concrete taken at layer $i$

$E_{ck}$  Secant elastic modulus of concrete taken at layer $k$

$E_{co}$  Elastic modulus of unstrained concrete

$E_f$  Elastic modulus of FRP

$f_c$  Stress in concrete

$f_{c'}$  Peak strength of concrete

$f_{ci}$  Stress in concrete in layer $i$

$f_{ck}$  Stress in concrete in layer $k$

$f_r$  Rupture stress of concrete

$H_{pile}$  Total depth of the trapezoidal GFRP sheet pile sections
\( i \) Summation of all layers from the bottom of the section to the elevation of interest

\( I_{xx} \) Second moment of inertia of the section about its x-x neutral axis

\( M \) Bending moment induced on the section

\( M_{cr} \) Bending moment at which concrete first cracks

\( M_i \) Bending moment at location or discrete region of interest along the length of the beam

\( M_{r \text{ (bond)}} \) Bending moment in section at time of ultimate failure due to bond

\( M_{r \text{ (bond-final)}} \) Bending moment in section at time of final bond failure

\( M_{r \text{ (bond-initial)}} \) Bending moment in section at time of initial bond failure

\( M_{r \text{ (flexure)}} \) Bending moment in section at time of flexure failure

\( M_{r \text{ (shear)}} \) Bending moment in section at time of shear failure

\( M_r \) Bending moment in section at time of failure

\( Q_i \) First moment of areas of all layers in the section either above or below the elevation or layer number \((i)\) of interest

\( t \) Thickness of a discrete layer within a section being modeled

\( t_{\text{flange}} \) Thickness of the flanges of the GFRP sheet pile section

\( t_{\text{web}} \) Thickness of webs of GFRP sheet pile section

\( V \) Vertical shear force

\( V_c \) Contribution of vertical shear strength provided by concrete

\( W_{\text{concrete}} \) Width of concrete present within a single discrete layer of the section being modeled

\( W_{\text{GFRP}} \) Width of GFRP present within a single discrete layer of the section being modeled
\( \alpha_1 \)  
Tension stiffening coefficient accounting for bond characteristics of reinforcement

\( \alpha_2 \)  
Tension stiffening coefficient accounting for sustained or repeated loading

\( \gamma_c \)  
Density of concrete

\( \delta \)  
Total mid-span deflection of the beam

\( \Delta v_i \)  
Change in deflection of beam within a discrete region of interest along the length of the beam

\( \Delta \theta_i \)  
Change in slope of beam within a discrete region of interest along the length of the beam

\( \varepsilon_c \)  
Strain in concrete

\( \varepsilon'_c \)  
Strain in concrete when stress equals \( f'_c \)

\( \varepsilon_{cc} \)  
Crushing strain of concrete

\( \varepsilon_{fc} \)  
Crushing strain of FRP

\( \varepsilon_{fr} \)  
Rupture strain of FRP

\( \varepsilon_r \)  
Rupture strain of concrete

\( \theta \)  
Angle of the webs of the GFRP sheet pile section measured relative to the vertical

\( \theta_i \)  
Slope of beam within a discrete region of interest along the length of the beam

\( \tau \)  
Shear stress

\( \tau_{ult} \)  
Ultimate bond strength at concrete/GFRP interface

\( \psi \)  
Curvature, measured as the strain gradient through a cross section

\( \Psi \)  
Curvature at location or discrete region of interest along the length of the beam
CHAPTER 1: INTRODUCTION

1.1 General

Conventional reinforced concrete structures are fabricated by casting concrete in temporary forms that are usually made from wood or steel. The forms are often held in place by temporary shoring or scaffolding structures. Upon hardening of the concrete, the forms and shoring are removed, revealing the concrete structure within. Stay-in-place formwork is a system that is not intended to be removed upon hardening of concrete for one or more reasons, often related to the speed and ease of construction. In this case, it is logical to design the system to provide a structural contribution, essentially as tension reinforcement. Open forms are particularly well suited for applications where positive bending is involved. This is because the inherent location of the formwork, at the bottom of the member, allows it to provide tensile reinforcement for resisting positive moments. This resistance, however, can only occur if sufficient shear connection exists between the concrete and the formwork.

The elimination of conventional rebar cages can significantly simplify the engineering and detailing process, as well as saving a great deal of time and effort during construction. It can also help to improve the longevity of the system by eliminating steel reinforcements that are susceptible to corrosion. Numerous corrosion-resistant stay-in-place open structural formwork systems have been explored in recent years (Cheng et al, 2006). They often utilize fibre reinforced polymer (FRP) materials. It should be noted that FRP materials still have a relatively short track record in structural applications, relative to steel. As such, their long term durability is yet to be confirmed. Since this type of formwork is typically mass produced using FRP pultrusion in factories, it is ready
to be used immediately when it arrives at the construction site. It is also relatively light weight, so forms can be shipped, maneuvered, and installed by a few workers without the aid of heavy machinery. Since these forms have excellent stiffness and dimensional stability, the need for scaffolding or shoring is greatly reduced or eliminated completely. In this case, the FRP formwork can simply be rested on supports at either end of the span, and then the concrete can be poured onto the formwork. In essence, the resultant member is a concrete/FRP hybrid member in which each of these materials is utilized efficiently; the FRP resists tension, and the concrete primarily resists compression. While the initial cost of FRP stay-in-place forms is likely greater than that of conventional concrete forms fabricated from wood or steel, this additional cost would be offset by improved ease and speed of erection, and reduced life-cycle costs of the overall structure due to superior longevity and durability.

1.2 Objectives

The primary objective of this study is to assess the performance of pultruded GFRP sections as stay-in-place structural open formwork for concrete flexural members, as well as to investigate some of the logistical details that arose while developing such a structural system. The main topics addressed by this study are:

1. Assessment of the performance of various adhesive and mechanical bond mechanisms at the concrete/FRP interface.
2. Studying both slab and girder systems using flat FRP sheets and trapezoidal FRP sheet pile sections, respectively.
3. Examining the various failure modes of slabs and girders.
4. Assessing the effect of introducing voids within the tension region of the concrete girder, essentially creating a box section to reduce self weight.

5. Developing an all-GFRP box girder by adhering flat pultruded GFRP sheets to the trapezoidal GFRP sheet pile sections. The box section was then used as a stay-in-place structural form for a thin concrete slab.

6. Studying the girder systems in both positive and negative bending to simulate continuity.

7. Compare the flexural performance of the hybrid concrete-FRP slab and girder systems tested in this study to conventional steel-reinforced slabs and girders.

8. Develop analytical models to help understand the structural behaviour of the slabs and girders, with an emphasis on bond behaviour at the FRP/concrete interface. The model is also used to study the following parameters for the girders: concrete strength ($f'_c$), bond strength ($\tau_{ult}$) at the concrete/FRP interface, angle of webs of the GFRP sheet pile section, thickness of the GFRP flanges, and width of the concrete slab over the sheet pile section.

1.3 Scope

The scope of this study includes experimental investigations and analytical models of the behaviour of flexural concrete members reinforced with GFRP stay-in-place structural open formwork.

The experimental investigation was intended to assess the feasibility of using commercially available, mass produced, pultruded GFRP sections as stay-in-place structural open forms for concrete slabs and girders. Eight one-way slabs and nine girders were constructed and tested in four-point quasi-static uniaxial monotonic bending.
These tests were used to quantify the performance of the concrete/GFRP hybrid system, and to investigate the performance of various bond mechanisms at the concrete/GFRP interface. The tests were also used to refine and optimize the cross-sectional geometry of such members by assessing the effects of several geometric variations such as the inclusion of a void within the tension region of the concrete, and the use of all-GFRP box girder sections.

The analytical model was developed to test theories and concepts pertaining to the complex mechanical behaviour of the aforementioned structural system. The model uses cracked section analysis, using a layer-by-layer technique in order to establish the flexural response of the member. It accounts for the non-linearity of the concrete in the section, and adopts conventional empirical tension stiffening theories. As the model synthesizes the flexural response of the member, it simultaneously checks various failure criteria such as flexural tension or compression failure, concrete shear failure by diagonal tension, and bond failure at the concrete/GFRP interface. Once the model was fully developed and verified, it was used in a parametric study to examine different types and ranges of parameters beyond the limitations of the experimental program.

1.4 Outline of Thesis

The contents of this thesis are listed below:

Chapter 2: A brief review of literature pertaining to the topics studied in this investigation.

Chapter 3: A manuscript that experimentally and analytically investigates concrete slabs reinforced with flat GFRP sheets using various bond mechanisms at the concrete/GFRP interface, various concrete thicknesses, and various spans.
Chapter 4: A manuscript that experimentally investigates concrete girders reinforced with trapezoidal GFRP stay-in-place structural open forms composed of commercially available sheet pile sections. Different bond mechanisms, cross-section configurations, and the effects of positive and negative bending are assessed.

Chapter 5: A manuscript that presents an analytical model of the girders discussed in chapter 4. The model is then used for a parametric study.

Chapter 6: Several conclusions that were drawn from this investigation, as well recommendations for further work in this area of research.

References

Appendix: A detailed description of the experimental method used, as well as the FORTRAN90 code used for some of the analytical models.
CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Fibre Reinforced Polymers (FRPs) have generated a great deal of interest in the structural engineering research field, due to their excellent physical and mechanical properties, including corrosion resistance, low self weight, and high tensile strength. The concept of structurally integrated formwork for concrete structures first emerged in 1901 when Sewell developed concrete-filled steel tubes (CFST) for columns and piles for offshore structures (Gardner and Jacobson, 1967). Concrete-filled FRP tubes (CFFTs) then emerged in the 1990s, taking advantage of FRP tubes as a noncorrosive alternative to steel tubes. In addition to the aforementioned qualities of FRP materials, CFFTs benefit from exceptional ease of construction because the FRP tube behaves as a stay-in-place structural form that replaces internal steel reinforcement. Under applied loads, the FRP tube acts as reinforcement that contributes to shear and flexural strength, in addition to providing the concrete core with confinement that can significantly improve compressive strength and prevent moisture intrusion. This synthesis of admirable qualities has made CFFTs an attractive system that has been researched mainly by two major groups (Mirmiran and Shahawy, 1997, and Fam and Rizkalla, 2001). This research has revealed many of the inherent advantages of FRP stay-in-place structural formwork, and has subsequently triggered interest in other types of FRP stay-in-place form systems. A comprehensive literature review on stay-in-place closed structural forms can be found in Chapter 9 of the ACI 440R-07 report.

Recently, there has recently been an interest in studying stay-in-place open forms. The term open formwork refers to formwork that does not completely enclose the
concrete within. This type of formwork is essentially a basin into which the concrete is poured. Stay-in-place open structural forms are particularly well suited for applications where positive bending is present. This is because the inherent location of the formwork (at the bottom of the member) allows it to provide tensile reinforcement for resisting positive bending moments. As a result, the majority of research performed on this topic has been related to slabs and beams.

This chapter generally reviews research that has been performed on stay-in-place open structural formwork for reinforced concrete flexural members. In particular, the chapter addresses FRP profiles that have potential as structural forms, various bond mechanisms, and flexural members that have been developed and studied. First, two conventional open form systems using steel and concrete materials are introduced.

2.2 Conventional Structural Form Systems

2.2.1 Prestressed Concrete Slab Forms

This system of open formwork comes in the form of very thin precast concrete panels, pretensioned using steel tendons, and is used in bridge deck applications. The panels are placed across the girders of the bridge, and concrete is then cast in place on the panels. The panels are designed to bear the weight of fresh concrete to avoid the need for shoring during construction. After the concrete has hardened, the system becomes fully composite. Chapter 16.2 of the PCI Bridge Design Manual (PCI, 2001) recommends a design procedure for implementing this technology. A newer version of this technology has been proposed (Badie et al, 1998) in order to improve composite action between slabs and bridge girders, as well as to improve the performance of the system in negative
bending applications. Clearly, the presence of steel prestressing tendons renders this system susceptible to corrosion.

### 2.2.2 Corrugated Steel Forms

Corrugated steel floor deck is essentially a stay-in-place open formwork for concrete slab applications. The corrugated shape of the steel sheet provides adequate flexural stiffness to bear the weight of the concrete before it hardens. Many steel decking systems also incorporate surface deformations so that a mechanical interlock may occur between the concrete and the steel form, thus resulting in composite action, which contributes to the flexural strength and stiffness of the concrete slab. Figure 2-1 is a cut-away drawing of a typical composite slab-on-girder floor.

Bailey Metal Products Ltd. has developed a steel stay-in-place structural open formwork system that is essentially an enlarged version of the aforementioned corrugated steel deck section. The product (Comflor Composite Floor System) is essentially a cold-formed stamped galvanized steel sheet that is trapezoidal in section, as shown in Figure 2-2. Multiple sections can be joined together in a tongue-and-groove fashion in order to create a large corrugated sheet of any size. The corrugations in this product feature a larger amplitude and wavelength than conventional corrugated steel decking. The sheet metal of these sections is also stamped with ribs running transversely on all surfaces (webs and flanges). The primary purpose of these ribs is to provide mechanical interlock between the steel forms and the concrete overlay in order to ensure that full composite action can be achieved. Upon completion, the system essentially takes the form of many reinforced concrete T-beams running parallel to one-another, as shown in Figure 2-2. It is designed for flooring, roofing, and decking applications. The steel section itself is stiff
enough that it can serve as a stable working platform prior to pouring concrete, and is capable of supporting the self-weight of fresh concrete without the aid of shoring.

It has been well established that the use of stay-in-place forms trap moisture within the concrete, which can cause harmful chloride attack leading to accelerated corrosion (Kuennen, 2006). Corrosion of conventional steel reinforcing bars can be structurally detrimental due to the loss of cross-sectional area, and spalling of concrete covers. Similarly, corrosion of steel open structural formwork, if not protected or galvanized, would cause reduction of cross-sectional area of the form, as well as a loss of the crucial bond between the steel form and the concrete overlay. Therefore, the use of FRP materials for these stay-in-place structural open forms could be a suitable alternative.

2.3 Flexural Behaviour of Pultruded FRP Sections

Several pultruded FRP sections are now commercially available and have the potential to be used as stay-in-place structural forms for concrete structures. Figure 2-3 shows some of these off-the-shelf sections of different configurations. Of particular interest is the trapezoidal section shown in Figure 2-3(e), which is produced as a sheet pile section. This section, as well as the flat sheet shown in Figure 2.3(a), are used in this study as stay-in-place forms. A series of flexural tests were performed on pultruded GFRP sheet pile sections in order to characterize their behaviour for use in waterfront retaining structures (Shao, 2006). The sections tested were not identical to those studied in this thesis, but are similar enough such that the test results could provide important information regarding failure modes and general flexural behaviour. Flexural and shear stiffnesses were determined using a multiple-span test method. The following equation describes deflection ($\delta$) with respect to shear stiffness ($GkA$), flexural stiffness ($EI$), load
(\(P\)), and span (\(L\)), where \(C_1\) and \(C_2\) are coefficients that are dependent upon the loading configuration.

\[ \delta = \frac{C_1}{EI} PL^3 + \frac{C_2}{GkA} PL \]  

(2-1)

Multiple spans and loading configurations were used in order to solve for the two coefficients, and subsequently determine the proportion of the deflection that was caused by shear deformation. Moment resistance was determined by applying a uniformly distributed load using an air bag. The profile was found to have a non-linear response, probably arising from the local buckling and flattening of the section. As a result, Equation 2-1 underestimated deflection at failure by 18%. Local buckling of the section was a significant problem. Transverse ties were used to prevent opening of the section. Under large deflections, the longitudinal stresses in a flexural member tend to produce a vertical component that compresses the cross-section of the member, due to large curvature. In essence, the extreme tension and compression fibres tend to be forced toward each other in regions of large curvature. This phenomenon is quite similar to ovalization exhibited by round hollow tubes undergoing bending (Ibrahim, 2000). As a result, the trapezoidal sheet pile section tends to flatten when experiencing large bending moments and curvatures. Consequently, the depth of the section would be reduced, which would effectively reduce the moment of inertia of the section. For this flattening to occur, the section must be able to expand laterally as the webs rotate outward. By installing horizontal ties, the sections were restrained from this expansion, and flattening was prevented. This flattening effect may also be prevented by placing multiple sheet piles side-by-side because they would restrain each other from lateral expansion.
2.4 Bond Mechanisms between Concrete and Structural Forms

In order to ensure that stay-in-place open structural forms contribute as tension reinforcement in flexure, it is crucial that an adequate shear connection exists between the concrete and the form. A study was carried out (Hall et al, 1998) on a hybrid concrete-FRP section incorporating FRP stay-in-place open structural form. Ribbed FRP sheets, as the one shown in Figure 2-3(b) produced as floor panels, provided tensile reinforcement, and behaved as permanent stay-in-place open structural formwork for the concrete slab overlay, as shown in Figure 2-4. The resultant concrete-FRP hybrid beams were tested in four-point bending. Initially, it was found that a significant amount of horizontal shear slippage occurred between the concrete and the FRP formwork. This severely limited the flexural capacity of the member since a large strain lag existed between the concrete and FRP. In an attempt to combat the aforementioned problem, adhesive bonding was used. This was accomplished by applying adhesive directly to the surface of the FRP formwork immediately prior to pouring concrete. This adhesive was specially formulated for bonding to fresh wet concrete. The resultant system behaved monolithically during positive bending. The observed mode of failure was diagonal tension shear cracking in the concrete, which can likely be attributed to the absence of shear reinforcement within the concrete, and the fact that the beams were designed to be over-reinforced. Tension cracking of concrete was severe in the vicinity of the longitudinal stiffening ribs of the FRP sheet. Overall, the concept of applying adhesive to the formwork prior to pouring concrete appears to have yielded positive results. It is an advantageous technique from a fabrication standpoint because it eliminates the need for mechanical shear studs or a bonded coarse aggregate coating on the surface of the formwork, both of which would be far more time consuming to install than an adhesive coating.
Another study was carried out (Dieter et al, 2002) on a hybrid concrete-FRP stay-in-place open structural formwork and FRP grid reinforcement for bridge deck applications. A pultruded FRP sheet stiffened by hollow FRP box sections, like the one shown in Figure 2-3(d), provided tensile reinforcement, and functioned as stay-in-place structural open formwork for the concrete slab overlay. A bi-direction grid composed of pultruded FRP elements provided the upper longitudinal and transverse reinforcement for regions of negative bending moments. A cut-away photograph of this system is shown in Figure 2-5. In order to generate sufficient shear bond between the FRP stay-in-place form and the concrete slab overlay, the surface of the FRP form was roughened prior to pouring the concrete by coating it with a mixture of epoxy and gravel. Due to the complex geometry of the formwork, this mechanism was only applied to horizontal surfaces. This was found to have a detrimental effect on the bond performance. In regions where the bond mechanism was absent, severe slippage occurred between the form and the concrete overlay. As a result, the flexural crack pattern in the concrete over unbonded regions was considerably more pronounced than it was in bonded regions.

A similar study (Bank et al, 2007) was carried out on concrete slabs cast on pultruded FRP planks resembling the section shown in Figure 2-3(b), which provided tensile reinforcement and acted as stay-in-place structural open forms. The bond mechanism that was provided between the concrete and FRP consisted of a rough coating comprising a mixture of epoxy and aggregates; however, the size of aggregates used was varied (both gravel and sand were used) in order to study the effect that this had on the flexural performance of the system. It was shown that the finer sand coating lead to a higher initial cracking moment than was achieved when the gravel coating was used. This study illustrated the feasibility of using the aforementioned FRP planks as stay-in-
place open structural forms for concrete slabs, and showed that the planks functioned as flexural reinforcement when the aggregate coating was employed. It was shown that the provisions of the ACI 440.1R-06 code can be used to accurately predict the flexural capacity of such structural systems; however, shear capacity can be more accurately predicted using the ACI 318-05 code. The technologies developed in this study were later used in a bridge that was constructed in Wisconsin in 2007, as shown in Figure 2-6, in order to create slabs across the main girders of the bridge.

2.4.1 Shear Strength of Adhesive Bond to Wet Concrete

Most adhesive systems intended for bonding to wet concrete have higher tensile and shear strengths than concrete. In fact, manufacturers of such adhesives will often claim tensile strengths in the order of 30MPa, and diagonal shear bond strengths (determined in accordance with ASTM C882) of approximately 10MPa. However, if a shear bond failure is to occur between a stay-in-place form and its overlying concrete, it is expected that this failure will occur slightly above the level of the adhesive through a plane within a thin layer of cement mortar, which is the weakest plane. The concrete is poured directly onto the adhesive; therefore, the adhesive is actually bonding to the concrete mortar. Consequently, it is unlikely that any coarse aggregates will intersect the shear failure plane. Therefore, the shear bond strength of this system will likely be equal to the shear strength of the cement mortar.

A study conducted on the shear behaviour of masonry bed joints indicates that the shear strength of mortar is extremely sensitive to confining stress (Pluijm, 1993). In this study, a confining stress of 1MPa yielded a shear strength of approximately 2MPa; whereas in the absence of any confining stress, a shear strength of approximately 1MPa
was observed. Another study (Hegemier et al, 1978) conducted on the behaviour of concrete masonry joints further supports the validity of these values. Figure 2-7 shows some relevant data yielded by these two studies. It is important to note that ASTM C882 inherently introduces normal confining stresses in conjunction with the desired shear stresses. It also allows strength to be calculated simply by dividing ultimate load by the cross-sectional area of the failure plane, without adjusting for the fact that the failure plane is not parallel to the direction of the applied load. Therefore, it is not surprising that adhesive manufacturers are capable of generating such high values of diagonal shear bond strength (10MPa) when using ASTM C882 tests.

In the context of horizontal shear between concrete and GFRP stay-in-place open structural forms in flexure, confining stresses are generally low and limited to the loading and supporting points. Therefore, the unconfined bond shear strength of any adhesively bonded system can be expected to be approximately 1MPa when coarse aggregates do not intersect the failure plane, thereby forcing the failure plane to pass through a thin layer of cement mortar slightly above the bond line.

2.5 Different Stay-in-Place Structural Open Formwork Systems

2.5.1 Hybrid FRP-Concrete Sandwich Panels

A sandwich panel is a flexural member consisting of a light-weight core that is sandwiched between two relatively stiff skins. The concept behind sandwich panels is that the skins are responsible for the longitudinal (tensile and compressive) stresses associated with flexure, whereas the light-weight core is responsible for the shear stresses involved. The core also acts as a spacer that separates the skins in order to increase the
depth of the section, thereby increasing the moment of inertia. This drastically increases flexural stiffness and strength with only a minimal increase in the mass of the specimen.

A study was carried out (Keller et al, 2006) on a hybrid FRP-concrete sandwich bridge deck panel. The tension skin of the panel comprised a commercially available pultruded GFRP decking sheet with integrated ribs (Figure 2-3(b)). The core of the panel was made from light weight concrete (densities of 900 and 1300 kg/m³ were used). The top compression skin of the panel was made from a thin layer of normal density concrete. The GFRP tension skin behaved as a stay-in-place open structural form for the two layers of concrete that were poured onto it. This was only possible because the specimen was designed for positive bending moments. Figure 2-8 is a rendering showing this sandwich panel section. These sandwich panels were tested in flexure using a three-point bending set-up. In some panels, the FRP tension skin was not bonded to the concrete. These panels attempted to rely upon mechanical interlock between the concrete and the ribs of the pultruded FRP decking panels in order to provide shear connection. However, because the ribs of the FRP panels were oriented longitudinally, this shear connection proved to be insufficient. As a result, significant slippage was observed between the light concrete core and the FRP tension skin, thus reducing composite action between these two materials. Consequently, tensile stresses within the light concrete core escalated, leading to flexural failure by tension cracking of the concrete. In order to counter these effects, some of the sandwich panels were fabricated using an adhesive bond between the light weight concrete core and the FRP decking panels. These panels failed by horizontal shear within the light concrete core itself prior to any bond failure occurring in the vicinity of the concrete/FRP interface.
The idea of utilizing lightweight materials in regions of low flexural stresses (near the neutral axis) is quite attractive because it can significantly reduce the self-weight of a member without an excessive loss of flexural stiffness or strength. However, it is important to recognize that the implementation of such a technology could shift the design to one that is limited by its shear strength.

2.5.2 Steel-Free Composite Slab on Girder

Typical composite slab on girder construction involves steel I-girders and corrugated steel floor decks that are overlain with a concrete slab. A similar system was investigated (Li et al., 2006) in which both the steel girder and form were replaced with GFRP sections of comparable dimensions. The system consisted of a pultruded GFRP I-beam (with unidirectional longitudinal fibres) overlaid by pultruded GFRP ribbed sheets (E-shaped sections) oriented horizontally. These ribbed sheets (Figure 2-3(c)) acted as permanent stay-in-place formwork for the concrete slab which was poured on top of the sheets, as shown in Figure 2-9. The ribs on these E-shaped sections were oriented transversely in order to provide adequate stiffness so that these sheets could bear the weight of the wet concrete. This also could potentially aid in providing the completed system with improved flexural strength and stiffness in the transverse direction; however, this characteristic was not studied. FRP bolts were used to connect the E-shaped sections to the I-beams. These bolts also behaved as shear studs in order to ensure monolithic composite action between the concrete slab and the FRP sections. The resultant composite girders were tested in four-point bending. Concrete strength and slab thickness were the primary test parameters varied; however, some specimens also included a laminate of carbon-FRP (CFRP) bonded to the bottom flange of the I-beam to improve
flexural stiffness and strength. The general mode of failure observed was horizontal shear cracking of the web of the GFRP I-beam. This proved to be a very brittle mode of failure. The specimens with thicker concrete slabs gave some warning of failure when the bottom of the concrete slab began to crack. This, however, was unrelated to the shear failure mode. It simply illustrated that the neutral axis in this specimen was located within the concrete slab. These cracks served as an indication that large deflections were occurring. The stiffness of the members was largely dictated by the thickness of the concrete slab. A relatively large percentage of deflections were caused by shear deformations occurring within the web of the girder. As a result, the CFRP layers provided very little contribution to stiffness. Also, since the specimens ultimately failed in shear, the addition of the CFRP layers yielded no increase in ultimate strength.

2.5.3 FRP Box Girder with Concrete in the Compression Zone

A novel FRP/concrete hybrid flexural member was proposed (Deskovic et al, 1995) in order to mitigate the shortcomings of structural members composed of these materials used independently. The new member is composed of a GFRP rectangular box section that is overlain with a concrete slab (Figure 2-10), and is intended for applications where positive bending is to be resisted. The concrete slab behaves as the compression flange of the member. The upper flange of the GFRP box section behaves as a stay-in-place form for the concrete slab, which simplifies the construction process considerably. Bond between the concrete and the GFRP section is facilitated by the application of a two-part epoxy adhesive prior to pouring the wet concrete. The hybrid member also included a CFRP laminate bonded to the bottom surface of the lower (tension) flange of the GFRP box section, to increase flexural stiffness. Also, because CFRP has a lower
failure strain than that of GFRP, the CFRP layer would fail prior to the tension flange of the GFRP section, thus providing warning signs of imminent flexural failure (pseudoductility). This is important since FRP and concrete are both brittle materials that do not provide obvious warning signs prior to failure. A number of potential failure mechanisms have been studied analytically for this hybrid member. The webs may buckle or fracture, resulting in shear failure; one of the beam’s elements could exhibit a flexural failure; the bond between the GFRP section and the concrete slab could fail; and the concrete slab could fail in diagonal shear. The most common mode of failure observed experimentally was debonding between the GFRP section and the concrete slab. Despite this unfortunate premature failure mode, the flexural response of the specimens showed good pseudoductility as a result of the CFRP laminate tension failure. This study, like the other ones presented earlier, highlighted the potential of hybrid members composed of hollow GFRP sections overlain with concrete slabs, but it also illustrated that such hybrid systems are highly dependent upon the quality of the shear bond between the concrete and the GFRP section.
Figure 2-1. Conventional composite slab-on-girder construction using corrugated steel form

Figure 2-2. Comflor Composite Floor System by Bailey Metal Products Ltd.
Figure 2-3. A selection of commercially available FRP sections

Figure 2-4. Concrete and GFRP ribbed sheet hybrid section
(Hall et al, 1998)
Figure 2-5. Bridge deck section with GFRP stay-in-place open form and FRP grid reinforcement (Dieter et al, 2002)

Figure 2-6. Installation of FRP planks as stay-in-place open structural forms for the deck of a bridge in Wisconsin (Bank et al, 2007)
Figure 2-7. Shear strength of cement mortar with varying amounts of confining stress ($\sigma$) (Pluijm, 1993, and Hegemier et al, 1978)

Figure 2-8. Sandwich panel comprising a GFRP ribbed sheet and concrete of two different densities (Keller et al, 2006)
Figure 2-9. Steel-free composite slab-on-girder design (Li et al, 2006)

Figure 2-10. Novel FRP/concrete hybrid flexural member (Deskovic et al, 1995)
CHAPTER 3: EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS OF GFRP STAY-IN-PLACE SYSTEM FOR CONCRETE SLABS

3.1 Introduction

This chapter provides experimental and analytical investigations into the behavior of one-way concrete slabs cast on glass-fiber reinforced polymer (GFRP) plates used simultaneously as formwork and as tension reinforcement. Eight slabs were fabricated and tested in four-point bending to investigate various bond systems and GFRP plate reinforcement ratios. Bond systems included fresh concrete adhesively bonded to the plate, coarse aggregates bonded to the plate prior to casting, and GFRP and steel mechanical shear connectors. The performance of these bond systems was objectively assessed and the failure modes were examined. The effect of altering the reinforcement ratio of the slab was examined by testing slabs of varying thicknesses while using a constant thickness of GFRP. A general analytical model has been developed to predict the full behavior and strength of the slabs, and was successfully verified. The model is able to detect various failure modes, including GFRP plate debonding. The performance of the slabs was compared with conventional steel reinforced concrete slabs of similar dimensions.

3.2 Experimental Program

The following sections provide details of test specimens and parameters, fabrication, instrumentation and test procedures of the concrete slabs.
3.2.1 Test Specimens and Parameters

Eight slabs, referred to as 1 to 8, were fabricated and tested. Table 3-1 provides a summary of the test matrix, including dimensions, spans, reinforcement ratios and the bond mechanisms. Slabs 1 to 4 were 1220 x 400 x 160mm, and were tested to compare different bond mechanisms: (a) concrete cast on wet adhesive, (b) coarse aggregates adhesively bonded to the GFRP plate prior to concrete casting, (c) GFRP shear connectors, and (d) steel shear connectors. Slabs 5 to 8 were 2440 x 400mm and their thicknesses varied. Slabs 5 to 7 all had a wet adhesive bond and their thicknesses varied from 110 to 210mm, which provided a GFRP reinforcement ratio ranging from 8.5 to 4.3 percent, respectively. This ratio is defined as the ratio between the cross-sectional area of the GFRP plate and concrete. It should be noted that because the GFRP plate used has a fixed thickness, the concrete slab thickness was varied to achieve the different reinforcement ratios (i.e. it was not possible to assess this parameter using the conventional technique of varying the amount of reinforcement in a cross-section of fixed dimensions). Slab 8 was quite similar to slab 6, except it utilized adhesively bonded coarse aggregates, also to examine different bond mechanisms in long slabs. The shear span-to-depth (a/d) ratio of slabs 1 to 4 was relatively small (2.4) (i.e. shear-bond critical), whereas (a/d) of slabs 5 to 8 ranged from 4.7 to 9.2 (i.e. flexural-bond critical).

3.2.2 Materials

3.2.2.1 Concrete – All specimens were fabricated using the same batch of concrete. The concrete was designed to have a compressive strength of 35MPa at 7 days, using high-early strength cement. The maximum aggregate size was 14mm, and slump was 152mm. Concrete cylinders, 102 x 203mm, were made from the same batch of concrete. At the
time of testing, the average concrete strength and standard deviation were 36MPa and 1.72MPa, respectively, determined in accordance with ASTM C39. More detailed statistical data on the concrete strength can be found in Table A1 of Appendix 1.

### 3.2.2.2 GFRP plates

Commercially available 9.5mm thick GFRP flat plates were received in a standard size of 1220 x 2440mm. The plates consisted of alternating layers of unidirectional E-glass roving and random mats, impregnated with polyester resin. Based on the manufacturer reported data (Figure 4-1(c)), the tensile strength and modulus in the longitudinal direction were 138MPa and 12.4GPa, respectively, and in the transverse direction were 69MPa and 6.9GPa, respectively. The bearing strength of the plate was 220MPa. Three plate bending tests were carried out by the author on 340 x 102mm coupons, to establish the longitudinal elastic modulus. The coupons were tested in four-point bending, with a span of 300mm and a distance between loads of 50mm as shown in Figure 3-1. Strains were measured on the tension and compression sides at mid-span. Based on these tests, the longitudinal flexural modulus was 18GPa, which is higher than the manufacturer’s specified elastic modulus by 45 percent. These flexural coupons failed by rupture of the outer most random mats of GFRP.

As discussed in Chapter 4, and to confirm this value, standard coupons for direct tension tests according to ASTM D3039/D3039M were also cut from these sections as shown in Figure 4-12. The central region of the coupons, where strains were measured, was 10mm wide and 50mm long. At either end of this central region, the coupons widened to 30mm (dog bone shape) in order to accommodate the grips of the loading machine and ensure that failure occurred within the central region. The total length of each coupon including the widened tabs and the narrower central region was 340mm.
Figure 4-14 shows the stress-strain curves of the coupons in tension. The results of these coupon tests showed similar elastic modulus (21GPa) to the flexural modulus determined from the aforementioned plate bending tests. In the absence of the tension coupon data at the time analysis was carried out, the modulus established from plate bending was used to model the slabs. This was shown to be lower by 12.6 percent than that established from tension tests. It should be noted, however, that in relatively thin concrete slabs with relatively thick GFRP plates as in this study, the FRP plate, although fully in tension, is likely subjected to a significant strain gradient. Therefore, it was assumed that the elastic modulus of these GFRP plates was equal to approximately 18GPa. More detailed statistical data on the results of the coupon tests for these GFRP sections can be found in Table A3 of Appendix 1.

3.2.2.3 Epoxy adhesives – Two types of epoxy adhesives were used. The first was used to bond freshly cast concrete to GFRP plates. It has low viscosity and hence can be applied like a paint. The specified tensile and shear bond strengths by the manufacturer were 34.5MPa and 10.3MPa, respectively. The second type of adhesives was used to bond coarse silica aggregates to the GFRP plate surface, prior to casting the concrete. It has a mortar-like texture with high viscosity, which made it ideal for bonding the coarse aggregates to the flat surface of the GFRP plates. The manufacturer’s data sheet specifies a concrete bond tensile strength of 4MPa, and a bond shear strength of 15MPa, based on failure within the outer concrete layer.

3.2.2.4 Coarse aggregates – Silica stones were used in the bond system and were 4 to 9mm in diameter.
3.2.2.5 Mechanical shear connectors – Both GFRP and steel studs were used. Threaded GFRP rods, 12.7mm diameter, were used, with an ultimate transverse shear resistance of 32.9kN, according to the manufacturer. The steel threaded rods used were 12.7mm diameter B7 steel, with yield and ultimate tensile strengths of 685MPa and 883MPa, respectively.

3.2.3 Fabrication of Slabs

GFRP plates were cut to size; their surfaces were cleaned thoroughly using acetone, and were then fitted with the designated bond system. For the wet adhesive bond, the surface was coated with a thin layer of epoxy resin using a conventional paint brush, as shown in Figure 3-2, no more than 45 minutes prior to casting the concrete on the GFRP plate while the epoxy was still wet. For the bonded coarse aggregates, a thick layer of the mortar-like adhesive was applied to the surface of the GFRP plates with a carefully controlled thickness and then a layer of the silica aggregates was applied and pressed into the adhesive. Once the adhesive was fully hardened, any loose aggregates were brushed off. This left an extremely rough surface that could interlock with the wet concrete, as shown in Figure 3-2. For the plates fitted with shear connectors, holes were drilled through the GFRP plates. For the GFRP connectors, shear studs were created by inserting the threaded rods into the holes, which were then fastened to the plate using special GFRP nuts on both sides of the plate. A similar installation method was used for the steel connectors, using steel nuts and washers. In both cases, the nuts were tightened to the manufacturer’s specified allowable torque in each case, in order to maximize friction between the nuts and the GFRP plates. The studs were arranged along two rows,
with a 100mm longitudinal spacing and 200mm transverse spacing. All studs were installed such that they protruded 50mm from the upper surface of the GFRP plate. The GFRP studs were designed such that the slab would have a comparable ultimate load to those adhesively bonded. Both the wet epoxy bonded- and aggregate-bonded slabs were anticipated to have similar strength as failure was expected to be by delamination within the concrete. Steel studs were designed to have similar size and spacing as GFRP studs; hence, they were expected to provide a higher bond strength.

After preparation of the GFRP plates, simple wooden wall systems were erected between each of the GFRP plates to form the sides of the slabs. A steel welded wire mesh with 3mm diameter wires spaced at 76mm was installed near the upper surface of the slab, mainly for shrinkage crack control. Concrete was then cast, and the surface was finished and moist-cured for at least 7 days.

3.2.4 Test Setup and Instrumentation

All specimens were tested in a four-point bending set-up, as shown in Figure 3-3. Slabs 1 to 4 were tested with a span of 1000mm and a constant moment zone of 250mm. Slabs 5 to 8 were tested with a span of 2200mm and a constant moment zone of 250mm. Load was applied in stroke control at a rate of 1mm per minute using a Riehle machine, with an integrated load cell. The slabs were instrumented with electrical resistance strain gages applied to the GFRP plate, 100mm displacement-type strain transducers applied to the GFRP plate and to the concrete on the compression side, and linear potentiometers (LPs) to measure mid-span deflection and any slip between the concrete and the GFRP plate at the ends.
3.3 Results of the Experimental Program

A summary of test results is provided in Table 3-2. Figure 3-4 shows the load-deflection, load-strain, and load-slip responses of short slabs 1 to 4. Figure 3-5 shows similar responses for longer slabs 5 to 8. However, because slabs 5 to 8 varied in concrete thickness, their normalized moment-curvature responses \( (M/bd^2) \) versus \( (\psi d) \) are also compared, where \( M, b, d \) and \( \psi \) are the moment, width, effective depth measured to the mid-thickness of the plate, and curvature of the slab. The curvature is calculated as the slope of the strain profile.

3.3.1 Effect of Bond Mechanism

Figure 3-4(a) compares the load-deflection responses of slabs 1 to 4. Slabs 1 and 2 with adhesive bond and bonded coarse aggregates behaved quite similarly and showed significantly higher stiffness than slabs 3 and 4 after cracking. In both slabs, horizontal shear slip was restrained by the continuously bonded surface. Consequently, all shear flow was distributed over the concrete-GFRP plate interface plane within each of the shear spans. Because these systems rely upon adhesive bonding, there was no progressive horizontal shear slip as the specimens were loaded to failure. This is evident by the insignificant slip measured at the ends (Figure 3-4(e)). Failure of slabs 1 and 2 was initiated by a diagonal shear crack, which propagated towards the concrete-GFRP plate interface. Once the bond shear strength was exceeded, this shear crack rapidly propagated horizontally along the interface, followed by a sudden and complete debonding at one shear span, as shown in Figure 3-6(a). By carefully examining the interface surfaces of slabs 1 and 2 after failure, it was concluded that bond failure occurred within a very thin layer of the cement mortar and not within the adhesive bond line. This was evident by a
visible layer of cement paste that remained bonded to the GFRP plate, as shown in Figure 3-6(c). It is worth noting that no discrete flexural cracks in slabs 1 and 2 were visible, yet a clear reduction in stiffness is observed at a load of about 60kN (Figure 3-4(a and b)). Therefore, it is believed that flexural cracking took the form of a network of well distributed very fine cracks, due to the excellent adhesive bond.

Slabs 3 and 4, with GFRP and steel studs, respectively, behaved quite differently from slabs 1 and 2. Both specimens exhibited a single flexural crack in the constant moment zone initially (Figure 3-6(a)), followed by the appearance of a horizontal debonding crack along the entire concrete-GFRP plate interface, as a result of slip after the loss of adhesion (Figure 3-4(c)). As the load increased, two more flexural cracks appeared in slab 3 and its stiffness was considerably lower than slabs 1 and 2 (Figure 3-4(a)). The flexural cracks occurred through vertical planes that pass directly through a pair of shear connectors, as would be expected. Eventually, all the GFRP threaded rods sheared off horizontally within one shear span, causing failure at a load comparable to those of slabs 1 and 2. In slab 4, although the stiffness after cracking was lower than slabs 1 and 2 due to some slip, it was higher than slab 3, (Figure 3-4(a)). This was a result of the higher elastic modulus of steel studs compared to GFRP studs, which led to significantly less slip (Figure 3-4(c)). As the steel studs did not fail, the slab achieved higher load than slab 3 and diagonal tension shear cracks appeared within each of the shear spans, similar to slabs 1 and 2. As the load increased, the shear crack opened further, and consequently slip at the concrete-GFRP plate interface increased. At the ends of the slab, vertical splitting cracks were observed along the planes of the two rows of steel studs, and eventually the load dropped. Slabs 1 to 4 failed well before developing the full tensile strength of the GFRP plate, as evident by the tensile strains at ultimate,
which were about 14 to 24 percent of the rupture strain (0.0135) observed in coupon tests (Figure 4-13(c) of Chapter 4).

Slabs 6 and 8 had a wet adhesive bond and bonded aggregates, respectively, similar to slabs 1 and 2, but with larger spans. Figure 3-5(a) shows that both specimens behaved similarly, with slab 8 being slightly stronger and stiffer. Both specimens exhibited several fine flexural cracks and then failed suddenly by debonding of the GFRP plate within one shear span as shown in Figure 3-6(b). The large diagonal tension shear cracks apparent in the figure are secondary and are consequent to the bond failure.

Figure 3-4(b) shows a distinctly different cracking behavior between slabs 1 and 2 with adhesive bond and 3 and 4 with mechanical shear connectors. The cracking load is slightly higher in the case of the adhesive bond than the mechanical bond. This is because the shear studs displace concrete, which creates a vertical plane of relative weakness in the concrete, making it easier for flexure cracks to be initiated within these planes. Also, the transition from the uncracked to cracked stiffness is accompanied by excessive deformations in slabs 3 and 4. This is attributed to the nature of the shear transfer mechanism, where no bond exists in between the studs, and hence, once a flexural crack occurred, some relative slip must have taken place before the shear connectors became fully activated. This initial slip explains the small reduction in load accompanied by excessive tensile strain in Figure 3-4(b) for slabs 3 and 4. It is also clear that the adhesive bond in slabs 1 and 2 has excellent tension stiffening characteristics, relative to the mechanical bond, because of the continuous bond surface, as evident by their insignificant slip (Figure 3-4(c)).
3.3.2 Effect of Reinforcement Ratio

In slabs 5 to 7, the reinforcement ratio was varied by having three different concrete thicknesses for the same GFRP plate (i.e. GFRP reinforcement ratios of 8.5, 5.7 and 4.3 percent, respectively), while the same wet adhesive bond system was used. The load-deflection responses (Figure 3-5(a)) can not be used for comparison, because of the differences in section size; therefore, the normalized moment-curvature responses (Figure 3-5(c)) are suitable for this assessment, as they reflect the effect of the single variable. It is evident from this plot that the flexural stiffness is proportional to the reinforcement ratio. A 33 percent increase in reinforcement ratio, from slab 7 to 6, resulted in a 22 percent increase in stiffness; whereas a 96 percent increase in reinforcement ratio, from slab 7 to 5, resulted in a 65 percent increase in stiffness. In this study it is not possible, however, to assess the effect of reinforcement ratio on flexural strength. This is because all specimens inherently failed by debonding, as shown in Figure 3-6(b), and not by GFRP rupture or concrete crushing. By carefully examining the strains at ultimate (Figure 3-5(b)), it is clear that the GFRP plate developed about 25 to 34 percent of its rupture strain (0.0135); however, slab 5 with the thinnest concrete profile had a concrete compressive strain of about 0.003, and as such, compression failure may have been imminent in this specimen according to ACI 318-05 code. No slip was observed in slabs 5 to 7 (Figure 3-5(d)).

3.4 Analytical Model

A model has been developed to predict the complete flexural behavior of one-way slabs reinforced by adhesively bonded FRP plates used as structural forms. The model was written in FORTRAN90, and incorporates three failure criteria that consider the
following possibilities: (a) flexural tension or compression failure, (b) concrete shear (diagonal tension) failure, and (c) bond failure. The model establishes the moment-curvature response of the section, which is then terminated at a point governed by one of the three failure criteria discussed above (i.e. the one producing the minimum load capacity).

3.4.1 Moment-Curvature Response

Conventional section analysis based on equilibrium of forces and strain compatibility is carried out, assuming full bond between the GFRP plate and the concrete, as follows:

Step 1: An initial strain is assumed at the extreme tension fiber of the section.

Step 2: The location of the neutral axis is also assumed in order to establish a complete strain profile.

Step 3: The stresses in the concrete and the GFRP are then determined at various layers throughout the depth of the section. For concrete, the following stress-strain function (Collins and Mitchell, 1997) was used in compression:

\[
f_c = f'_c \left( \frac{n \left( \frac{e_c}{e'_c} \right)}{n-1 + \left( \frac{e_c}{e'_c} \right)^{nk}} \right) \]

\[
n = 0.8 + \frac{f'_c}{17}
\]
$k = 0.67 + \frac{f_c'}{62}$ \hspace{1cm} (3-1)

where $f_c'$ is the compressive strength of concrete, $f_c$ is the stress at a given strain $\varepsilon_c$, and $\varepsilon_c'$ is the strain in concrete at $f_c'$. While there are several other similar empirical models for the non-linear behaviour of concrete in compression, the selected stress-strain function is widely accepted among civil engineering researchers. The strain $\varepsilon_c$ is limited to 0.003 (ACI 318-05) or 0.0035 (CAN/CSA-A23.3-94). For concrete in tension, a linear stress-strain relationship was assumed until cracking. This was followed by a tension stiffening function (Collins and Mitchell, 1997) as follows:

$$f_c = E_{co} \varepsilon_c \hspace{1cm} \text{(for } f_c \leq f_r)$$

$$f_c = \frac{\alpha_1 \alpha_2 f_r}{1 + \sqrt{500(\varepsilon_c - \varepsilon_r)}} \hspace{1cm} \text{(for } f_c > f_r) \hspace{1cm} (3-2)$$

where $E_{co} = 4750 \sqrt{f_c'}$ (ACI 318-05), the modulus of rupture $f_r = 0.62 \sqrt{f_c'}$ (ACI 318-05), and $\varepsilon_r$ is the strain at cracking. $\alpha_1$ is a tension stiffening factor accounting for bond characteristics of the reinforcement and is taken as 1.0 to model the zero-slip characteristics of adhesive bonding. $\alpha_2$ is a tension stiffening factor accounting for sustained or repeated loading, taken as 1.0 for short term monotonic loading.

**Step 4:** The stresses are integrated with the areas of their respective layers and summed in tension and compression. If the sum is not equal to approximately zero, then a new value of the neutral axis depth is assumed in step 2 and the process is repeated, until equilibrium is satisfied.
Step 5: The sum of the moments generated by all forces ($M$) is calculated and the curvature ($\psi$) is also calculated as the slope of the strain profile for the particular strain assumed at step 1. This provides one point on the ($M$-$\psi$) response. The strain at the extreme tension fiber is then increased slightly in step 1 and the process is repeated, to establish the full ($M$-$\psi$) response.

3.4.2 Failure Criteria

In order to model the following failure criteria, the aforementioned moment-curvature algorithm was repeated with the exclusion of tension stiffening in the concrete to establish the termination (failure) point. The purpose of this omission was to account for the fact that both flexure and bond failure are likely to occur at the location of a crack.

3.4.2.1 Flexure

The process of developing the moment-curvature response continues until the extreme concrete strain in compression reaches 0.003 or the GFRP extreme tensile strain reaches the rupture strain. The corresponding moment in this case defines the ultimate value, governed by flexural strength $M_{r(flexure)}$.

3.4.2.2 Diagonal Tension Shear

For a given slab with a specific span and loading system, the possibility of shear failure of the concrete by diagonal tension is checked. For the moment ($M$) calculated for each strain increment in the process of establishing the moment-curvature response, the corresponding shear force diagram of the slab is established, from which the critical shear force ($V_c$) is determined. The concrete contribution to the shear resistance ($V_c$) is then
calculated using available code provisions, namely ACI 440.1R-06 and CAN/CSA-S806-02 (both for concrete structures reinforced by FRP):

\[
ACI \text{ 440.1R-06: } V_c = \frac{2}{5} \sqrt{f'_c} B c
\]  

\[
\text{CAN/CSA-S806-02: } V_c = 0.2 \sqrt{f'_c} B d
\]

where \( B \) is the slab width, \( c \) is the depth of the compression region of the section (above the neutral axis) (mm), and \( d \) is the depth of the section to the centroid of the GFRP plate (mm).

As will be discussed in the verification section, Equation 3-3 was found to be excessively conservative, whereas Equation 4 showed very good agreement with the experimental results. Once the vertical shear force \((V)\) is equal to \((V_c)\), shear failure is assumed to have occurred and the corresponding mid-span moment is \(M_{r\text{(shear)}}\).

### 3.4.2.3 Bond

This section provides an analytical approach to predict the load at which bond failure occurs. This approach involves the layer-by-layer process used earlier to establish the moment-curvature response, applied here at various sections along the span. The horizontal shear stress \((\tau)\) at the level of concrete-GFRP plate interface can be calculated using the following fundamental equation of mechanics:

\[
\tau = \frac{VQ}{I_{xx} B}
\]  

where \( V \) is the total vertical shear force experienced by the beam at a given cross section, \( Q \) is the first moment of area of the GFRP plate about the neutral axis of the cross-section of the slab, \( I_{xx} \) is the second moment of area of the section about its neutral axis, and \( B \) is
the width of the slab (bonded width). Prior to cracking, this is a fairly simplistic calculation. However, once flexural cracking occurs, the second moment of area of the section is reduced, and the neutral axis of the section begins to shift. Therefore, at any given load, the values of $I_{xx}$ and $Q$ are only constant within the uncracked region, where the bending moment ($M$) is less than the cracking moment ($M_{cr}$); however, they vary substantially in the cracked zone. Within the cracked regions, $I_{xx}$ and $Q$ are calculated on the basis of a cracked section. This is to account for the fact that debonding is typically initiated at the location of a crack (i.e. $\tau_{max}$ is reached at a crack). As the load and moment increase, the location where $M_{cr}$ is reached shifts closer to the supports.

Figure 3-7(a) shows conceptual distribution plots of $V$, $M$, $I_{xx}$, and $\tau$, for a simply supported slab undergoing four-point bending with an arbitrary load that generates bending moments exceeding ($M_{cr}$). From this plot, it is clear that the points of maximum horizontal bond shear stress ($\tau$) are directly beneath the applied loads. Therefore, in order to capture the load at which initial debonding occurred, it was necessary to monitor the changing values of $M$, $I_{xx}$, $V$, $Q$, and $\tau$ at these points. The neutral axis location was first established. By summing the second moment of the areas of each discrete layer of the section about the neutral axis, it was possible to establish a value of $I_{xx}$ for the cracked section at any given bending moment. Similarly, summing the first moment of the areas of each discrete layer within the GFRP sheet about the neutral axis of the total section gave a value of $Q$ for the section at any given moment.

It is important to note that because of the concrete non-linearity, the Young’s modulus is not constant throughout the depth of the section. Therefore, while performing these calculations, it was important to transform the widths of each layer ($B_i$) by the modular ratio of that particular layer, relative to the modulus of unstressed concrete.


\( \frac{E_{ci}}{E_{co}} \), where \( E_{ci} = \frac{f_{ci}}{\varepsilon_{ci}} \), \( f_{ci} \) is the stress at layer \( i \), corresponding to \( \varepsilon_{ci} \) of the layer and is calculated from Equations 3-1 or 3-2, and \( E_{co} \) is the initial modulus as defined earlier. This means that for layers within the concrete, the modular ratio varied depending upon the level of strain experienced by that particular layer. For the layers within the GFRP plate, the modular ratio is a constant value \( \frac{E_f}{E_c} \), where \( E_f \) is the modulus of GFRP, regardless of the strain level, since GFRP is a linear material.

For each value of \( M \) in the cracked section analysis, the values of \( I_{xx}, V, Q \), and \( B \) were used to calculate \( \tau \) at the concrete-GFRP plate interface, using Equation 3-5. When the maximum \( \tau \) value reaches the ultimate bond shear strength \( \tau_{ult} \) (discussed later), debonding is initiated at this point (Figure 3-7(b)). The corresponding mid-span moment \( M_r \) (bond-initial) is calculated and stored. Because of the loss of bond stress in the regions where debonding has occurred, the bond shear stress in all remaining bonded regions of the shear span must be increased in order to make up for this loss of horizontal shear flow. Consequently, the debonded regions begin to increase in length, propagating outwards toward the supports (Figure 3-7(c)). Eventually, they will extend to the points in the slab that experiences \( M_{cr} \). Beyond these points, the value of \( \tau \) is constant (uncracked section); therefore, the bond stress is uniformly distributed between the point of \( M_{cr} \) and the end of the overhanging region beyond the support (Figure 3-7(d)). In order to calculate the value of the constant bond stress \( \tau \) in the uncracked region, it was necessary to determine the total tension force in the GFRP plate within the unbonded region. Within this region, the GFRP strain, stress, and force are all constant, and their values were taken to be equal to the average of those values found at either end of the unbonded regions. Although the localized unbonded region does not abide by the assumptions upon which cracked-section-analysis is based, the bonded regions.
surrounding this region are fully intact and behave in a manner consistent with the cracked-section-analysis. Figure 3-7(d) illustrates the assumed distribution of tensile strains in the GFRP sheet. The abrupt shifts in tensile strain can be justified by shifts in the slab’s neutral axis. The value of constant bond stress $\tau$ can then be calculated by dividing the value of the tension force in the GFRP plate within the unbonded regions by the total area of the concrete-GFRP plate interface within the uncracked region of one shear span. Once this value of $\tau$ reaches $\tau_{ult}$, a complete debonding failure occurs. The corresponding mid-span moment $M_r_{(bond,final)}$ is calculated and stored. The larger value of $M_r_{(bond,initial)}$ and $M_r_{(bond,final)}$ governs for bond failure and is considered $M_r_{(bond)}$.

In the tested slabs with wet adhesive bonding or bonded aggregates, it was noted that a thin layer of cement paste was adhered to the GFRP plate after debonding. As such, failure was governed by the shear strength $\tau_{ult}$ of the cement paste and not by the adhesive. A typical value reported in literature for the shear strength of unconfined conventional cement mortar is 1MPa (Hegemier et al, 1978, and Pluijm, 1993). Therefore, for the purpose of this investigation, the ultimate bond shear strength is assumed to be 1MPa.

### 3.4.3 Validation of Model

This model was developed for adhesive bonded slabs; therefore, it has been validated using the results of slabs 1, 2, and 5 to 8. Figure 3-8 presents the experimental versus predicted moment-curvature responses of the specimens, which shows good agreement. Table 3-3 provides a summary of the predicted moment capacities based on the various failure modes, namely $M_r_{(flexure)}$, $M_r_{(shear)}$, and $M_r_{(bond)}$. The lowest of the three moments governs. The ultimate strength of slabs 1 and 2 was governed by shear failure.
of concrete, whereas in all other specimens the model predicted bond failure, similar to the experimental failure modes. For slabs 1 and 2, which failed in shear, the predicted $V_c$ values using the ACI 440.1R-06 and the CAN/CSA-S806-02 code equations varied substantially and were 11 and 28kN, respectively. The CAN/CSA-S806-02 predicted value agreed best with the experimental results. The ultimate load of slab 2 was underestimated by the model by 7.5 percent, whereas the ultimate load of slab 1 was predicted quite accurately to within 0.4 percent, even though both specimens failed in shear. Also, slab 6 with wet adhesive bonding, failed at a slightly lower load than slab 8 with bonded aggregates; and the model prediction, which is the same for both, was within 24 percent of the actual failure load of slab 6, and within 0.4 percent of the actual failure load of slab 8.

3.4.4 Model Application to other Load Cases

In the previous sections, the model was illustrated and verified using slabs tested in four-point bending. The same methodology and concepts can be adopted for any load case. Figure 3-9 illustrates conceptual plots of $M$, $I_{xx}$, $V$, and $\tau$, for a simply supported slab with an arbitrary uniformly distributed load that generates bending moments exceeding the slab’s cracking moment. It is evident from this illustration that there is more than one location along the length of the slab at which the bond may first fail (i.e. two peaks), therefore, both values ($\tau_{\text{max}1}$ and $\tau_{\text{max}2}$) must be checked against $\tau_{\text{ult}}$.

3.5 Comparison with Conventional Reinforced Concrete Slabs

In this section, an attempt is made to compare the moment-curvature responses of concrete slabs with GFRP stay-in-place forms, namely slabs 5 and 7 of this study, to
conventional steel-reinforced concrete (RC) slabs of the same concrete strength and similar or comparable outer dimensions. The objectives are to establish the steel reinforcement ratios required to achieve the same moment at ultimate (i.e. when failure occurred due to debonding of the GFRP plate) and same flexural stiffness of the GFRP-reinforced slabs. Conventional cracked section analysis has been used to obtain the moment-curvature responses of the RC slabs. This analysis was performed using computer software called Response2000, which was developed by Evan Bentz and Professor Michael P. Collins at the University of Toronto. Steel rebar of 400MPa and 600MPa yield and ultimate strengths, respectively, has been used in this analysis. Provisions of ACI 318-05 regarding concrete cover and limitations on rebar spacing have been satisfied. For cast-in-place concrete slabs exposed to weather, a 40mm concrete cover has been used for 16M bars and smaller. Clear spacing between bars was at least equal to the bar diameter, and not less than 25mm.

For comparison with slab 5, an attempt was first made to design section RC5 with outer dimensions identical to slab 5 and using 16M bars. However, this design failed to match the flexural capacity and stiffness of slab 5, given the concrete cover and rebar spacing limitations. Therefore, alternative sections with a slightly increased thickness were designed, namely section RC5a with a 116mm thickness for a similar stiffness and section RC5b with a 129mm thickness for similar strength to slab 5. It was found that 7.1 percent and 6 percent steel reinforcement ratios are required in sections RC5a and RC5b to achieve similar flexural stiffness and strength to slab 5, respectively. Figure 3-10(a) shows the moment-curvature responses of the slabs. These sections were the thinnest possible slabs that could be designed with comparable performance to slab 5. This design has lead to over-reinforced sections, where concrete crushes before the steel rebar yields.
For comparison with slab 7, sections RC7a and RC7b were designed to have the same outer dimensions as slab 7. Sections RC7a and RC7b required 1.1 percent and 1.8 percent steel reinforcement ratios to match the flexural stiffness and strength of slab 7. Figure 3-10(b) shows the moment-curvature responses of these slabs. In this case both RC7a and RC7b achieved a ductile failure, where the steel has yielded before the concrete crushed.

It is clear from Figure 3-10 that a relatively large steel reinforcement ratio could be necessary to match the flexural stiffness and strength of the GFRP-reinforced slabs, particularly slab 7. Also, the RC sections that have a similar strength have higher stiffness, whereas those having similar stiffness have lower strength than the GFRP-reinforced slabs. It is interesting to note that, due to the required concrete cover for steel reinforcement, the effective depths of slabs 5 and 7 are considerably greater than the effective depths of their conventional RC counterparts. This is a testament to the efficiency gained by using FRP structural forms.
Table 3-1. Summary of test matrix

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>Length (mm)</th>
<th>Span (mm)</th>
<th>Shear span (mm)</th>
<th>Width (B mm)</th>
<th>GFRP plate thickness (mm)</th>
<th>Total thickness (Rounded) (h mm)</th>
<th>Effective depth (d mm)</th>
<th>a / d</th>
<th>Reinf. ratio ρ (%age)</th>
<th>Bond mechanism</th>
</tr>
</thead>
<tbody>
<tr>
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<td>400</td>
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<td>160</td>
<td>157</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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Table 3-2. Summary of test results

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<th>Spec. ID</th>
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<th>Peak moment (kNm)</th>
<th>Strain at peak load</th>
<th>Load at first slip (kN)</th>
<th>Slip at peak load (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>Top (micro)</td>
<td>Bottom (micro)</td>
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<td></td>
</tr>
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<td>4236</td>
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Concrete shear – bond failure
Shear failure of GFRP studs
Excessive slip of steel studs
Bond failure
Bond failure
Bond failure
Bond failure

Table 3-3. Summary of model predictions

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<td>M_r (shear)</td>
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<td>28</td>
</tr>
<tr>
<td>2</td>
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<td>7</td>
<td>104</td>
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</tbody>
</table>

\(^a\) First bond failure  \(^b\) Ultimate bond failure
Figure 3-1. GFRP plate bending test

Figure 3-2. Fabrication of slab test specimens

Figure 3-3. Test setup
Figure 3-4. Test results of slabs 1 to 4
Figure 3-5. Test results of slabs 5 to 8
Figure 3-6. Failure modes

(a) Slabs 1 to 4
(b) Slabs 5 to 8
(c) Interface surface texture after failure of slabs 1 and 2
Figure 3-7. Progression of interface bond stress distribution as load increases in four-point bending
Figure 3-8. Experimental versus predicted moment-curvature responses

Figure 3-9. Progression of interface bond stress distribution under uniform load
Figure 3-10. Comparison between slabs 5 and 7 and conventional RC slabs
4.1 Introduction

This chapter provides an experimental investigation into concrete girders fabricated using commercially available pultruded GFRP sections originally used as sheet piles. The GFRP sections are used as formwork, while being the sole tension reinforcement for the girders. Similar to the slabs discussed in Chapter 3, various adhesive and mechanical bond systems are investigated. The effect of implementing a void within the tension region of the concrete is explored. Another configuration composed of an all-GFRP trapezoidal box girder, in which concrete is only used as a compression flange, is developed. The aforementioned systems are studied for applications in positive and negative bending, to simulate continuous beams.

4.2 Experimental Program

4.2.1 Test Specimens

Nine girders, referred to as girders 1 to 9, were fabricated using trapezoidal pultruded GFRP sheet pile sections as stay-in-place structural open formwork. The cross-sectional configuration of the sheet pile, which was fabricated by Creative Pultrusions Inc, is shown in Figure 4-1(a and b) and Figure 4-2(a). The specific section used was SuperLoc 1610. All girders had a total length of 3350mm.
4.2.1.1 Geometric Configurations

Table 4-1 provides a summary of the test specimens. The specimens were divided into three geometric types. The first type (Type 1) included girders 1, 2, and 3 which were cast with a void in the tension region of the concrete, as shown in Figure 4-3, and had a 70mm thick concrete slab above the upper flanges of the sheet pile section. The void size was 245x150mm such that the ceiling of the void was at the same level as the upper flange of the sheet pile, and concrete webs were still provided for shear resistance. This design was selected since the neutral axis of the section is located slightly above the void; therefore, the void only displaced concrete within the tension region. A 250mm thick diaphragm of a solid concrete fill was provided at both ends of the girders in order to prevent web crippling over the supports. All three specimens were tested in positive bending. The main parameter studied using these specimens was the bond mechanism, where three different bond systems were compared, as discussed later.

The second type of specimens (Type 2) included girders 4 and 5 which were completely filled with concrete, as shown in Figures 4-4 and 4-5. These specimens consisted of the pultruded GFRP sheet pile sections overlain with concrete. Similar to girders 1 to 3, the concrete depth was controlled such that a 70mm thick concrete slab was provided above the upper flanges of the sheet pile section. Girders 4 and 5 were identical; however, girder 4 was tested in positive bending while girder 5 was tested in negative bending to simulate the behaviour at an internal support of a continuous beam. Figure 4-6 shows a photograph of girder 4.

The third type of specimens (Type 3) were unique in that they were composed of built up all-GFRP box sections. These box section girders 6, 7, 8, and 9 were created by bonding flat 9.5mm thick pultruded GFRP sheets, similar to those used for the slabs in
Chapter 3 (information is given in Figure 4-1(c)), across the upper flanges of the sheet pile sections, resulting in a trapezoidal hollow box section, as shown in Figures 4-7 to 4-10. Girders 6 to 8 incorporated a concrete slab on top of the flat upper sheet of the box section. These slabs were 60mm thick so that the overall depth of these specimens was equal to that of the Type 1 and Type 2 specimens. Figure 4-11 is a photograph showing the cross-section of girders 6, 7, and 8. Girder 9 did not have a slab and was tested to establish the flexural stiffness and strength of the stay-in-place box section. Girder 9 was first tested in negative bending (9b) up to a relatively small load and then unloaded to establish the stiffness. It was then tested again to failure in positive bending (9a). Girders 6 and 7 were both tested in positive bending to compare two different bond systems (discussed later). Therefore, these specimens incorporated a 250mm thick diaphragm of solid grout within the box section over each of the supports. Girder 8 was tested in negative bending. Therefore, it incorporated a 500mm thick diaphragm of solid grout within the box section between the loading points. Girder 9, which was tested in both positive and negative bending, incorporated a 250mm thick diaphragm of solid grout within the box section over each of the supports.

4.2.1.2 Bond Mechanisms

Prior to pouring concrete into the forms, each of the GFRP sections was fitted with a mechanism designed to resist horizontal shear-slip between the concrete and the GFRP section. Four discrete bond systems were investigated, most of which are similar to the slabs in Chapter 3. Mechanism M1 involves a liquid epoxy adhesive applied directly to the surface of the GFRP form immediately prior to pouring the concrete. Mechanism M2 involved applying a rough layer to the surface of the GFRP sections in order to create
mechanical interlock between the concrete and the GFRP. This was accomplished by applying a thick mortar-like adhesive to the surface of the GFRP sections, and then applying a layer of 4 to 9mm diameter silica pebble aggregates onto the adhesive. Mechanism M3 utilized shear studs. These studs were created by drilling holes through the upper flat surfaces (flanges) of the GFRP sections, passing threaded steel rods through these holes, and then fastening the rods in place with a nut on either side of the GFRP section. Mechanism M4 was similar to mechanism M3 with the addition of heads on the studs. These heads were created by screwing a nut onto the top of each stud, and then jamming these nuts in place by distorting the threads with a hammer and punch. As shown in Table 4-1, girders 1, 2, and 3 were fitted with bond systems (M1), (M2), and (M3 combined with M1), respectively. Girders 4, 5, and 6 were fitted with the (M1) bond system. Girders 7 and 8 were fitted with (M4) combined with M1. Girder 9 was fitted with the (M3) studs. In girders 6, 7, 8, and 9, the same thick mortar-like adhesive used in bond system (M2) was also used to bond the flat GFRP plate to the sheet pile section in order to create the box section.

4.2.1.3 Parameters Studied

Girders 1, 2, and 3 were compared and girders 6 and 7 were compared in order to study the different bond mechanisms used at the concrete/GFRP interface. Girders 1, 4, and 6 were compared and girders 3 and 7 were compared in order to study different cross-sectional configurations. Girders 4 and 5 were compared and girders 7 and 8 were compared in order to assess the performance of the system under negative bending conditions and to compare this performance with its performance in positive bending. Girder 9 was tested in order to assess the performance of the box girder in the absence of
concrete, and was compared with girders 7 and 8 in order to quantify the contribution of
the concrete slab to the behaviour of the composite system in both positive and negative
bending.

4.2.2 Materials

4.2.2.1 Concrete – All concrete batches were ordered to have a compressive strength of
35MPa at 7 days, using high-early strength cement. The maximum aggregate size was
14mm and slump was 152mm. Concrete cylinders, 102 x 203mm, were made from the
same batch of concrete. For girders 1 to 3, the concrete compressive strength ranged from
33MPa to 39MPa with an average of 36MPa, based on testing cylinders in accordance
with ASTM C39. For girders 4 and 5, the concrete compressive strength ranged from
33MPa to 40MPa with an average of 37MPa, based on testing cylinders. For girders 6
through 8, the concrete strength ranged from 44MPa to 58MPa with an average of 52MPa
based on cylinder tests. More detailed statistical data on the concrete strength can be
found in Table A1 of Appendix 1.

4.2.2.2 Grout – Each of the four Type 3 girders 6 to 9 incorporated diaphragms of solid
grout within their hollow GFRP box sections. The grout used was made by the Sika
Corporation and is called SikaGrout 212. Grout cylinders, 102 x 203mm, were made
from the same batch of grout used in the specimens. The average compressive strength
was found to be 36MPa in accordance with ASTM C39. More detailed statistical data on
the grout strength can be found in Table A2 of Appendix 1.
4.2.2.3 GFRP Sheet Pile Sections – Commercially available sheet pile sections were fabricated by Creative Pultrusions Inc. These sections, known as SuperLoc 1610 (Figure 4-1(a and b)), have a trapezoidal shape with a depth of 254mm and a width of 609.6mm; the sections also incorporate pin and eye connection details along their longitudinal edges so that multiple sections can be joined side-by-side, resulting in a wide corrugated section. For the purposes of this investigation, each girder specimen incorporated a single SuperLoc 1610 section oriented longitudinally. The upper and lower flanges of these sections are 8.89mm thick. The webs, which are inclined 20 degrees from the vertical, are 7.62mm thick. The sheet pile sections are mass produced with a proprietary pultruded GFRP layup. The material properties of this layup, however, are available from a manufacturer’s data sheet (Figure 4-1(a and b)). It should be noted that Types 1 and 2 specimens (girders 1 through 5) were fabricated using sections comprising a slightly different resin matrix (polyester) from the Type 3 specimens (polyurethane). For the sheet piles used in Type 1 and 2 specimens (girders 1 to 5) polyester resin was used, and the manufacturer reported data indicates longitudinal tensile strength and modulus of 276MPa and 26.2GPa, respectively, and longitudinal compressive strength and modulus of 241MPa and 26.2GPa respectively (Figure 4-1(a)). For the sheet piles used in Type 3 specimens (girders 6 to 9) polyurethane resin was used, and the manufacturer reported data indicates longitudinal tensile strength and modulus of 517MPa and 26.2GPa, respectively, and longitudinal compression strength and modulus of 345MPa and 26.2GPa respectively (Figure 4-1(b)).

The sections were each cut down to a length of 3350mm, which was the total length of the girder specimens. Coupons were cut from the flanges of these sections as shown in Figure 4-12. The central region of the coupons where strains were measured
was 10mm wide and 50mm long. At either end of this central region, the coupons widened to 30mm in order to accommodate the grips of the loading machine and ensure that failure occurred within the central region. The total length of each coupon including the widened tabs and the narrower central region was 340mm. The coupons were tested in accordance with ASTM D3039/D3039M. Figures 4-13(a and b) show the stress-strain curves of the coupons in tension for the material used in girders 1 through 5, and girders 6 though 9, respectively. It should be noted that some of the strain gauges on these coupons failed prior to the coupons reaching their ultimate tension failures. As such, these figures have been labeled with the stress at ultimate failure for each coupon. The elastic modulus obtained experimentally using these coupon tests was quite consistent with the manufacturer’s specified elastic modulus of 26.2GPa; however, tensile strengths from the manufacturer were underestimated. More detailed statistical data on the results of the coupon tests for these GFRP sections can be found in Table A3 of Appendix 1.

4.2.2.4 GFRP plates – Commercially available 9.5mm thick GFRP flat plates, similar to those used in the slabs of Chapter 3, were received in a standard size of 1220 x 2440mm. The plates were fabricated by Creative Pultrusions Inc., and the sections used were entitled Pultex 1500 Flat Sheet (Figure 4-1(c)). These plates were to be used as the upper flanges of the box girders for girders 6 through 9 (Type 3 specimens). The plates consisted of alternating layers of unidirectional E-glass roving and random mats, impregnated with polyester resin. Based on the manufacturer reported data, the tensile strength and modulus in the longitudinal direction were 138MPa and 12.4GPa, respectively, and in the transverse direction were 69MPa and 6.9GPa, respectively. As discussed in Chapter 3, three plate bending tests were carried out on 340 x 102mm
coupons, to establish the longitudinal elastic modulus. The coupons were tested in four-point bending, with a span of 300mm and a distance between loads of 50mm as shown in Figure 3-1. Strains were measured on the tension and compression sides at mid-span. Based on these tests, the average longitudinal flexural modulus was 18GPa, which is higher than the manufacturer’s specified elastic modulus value by 45 percent. These flexural coupons failed by rupture of the outer most random mats of GFRP. To confirm these results, coupons for direct tension tests were also cut from these sections as shown in Figure 4-12, following the same procedures as used for the coupons from the sheet pile sections. Figure 4-14 shows the stress-strain curves of the coupons in tension. The results of these coupon tests showed a similar elastic modulus to the flexural modulus determined from the aforementioned plate bending tests. In the absence of the tension coupon data at the time analysis was carried out, the modulus established from plate bending was assumed to be accurate, as with the slabs. More detailed statistical data on the results of the coupon tests for these GFRP sections can be found in Table A3 of Appendix 1.

4.2.2.5 Epoxy adhesives – Two types of epoxy adhesives were used. The first is used to bond freshly cast concrete to the GFRP sections. It has low viscosity and hence can be applied like a paint as discussed in Chapter 3. The second type was used to bond coarse silica aggregates to the GFRP plate surface prior to casting the concrete, as well as to bond the GFRP plate to the sheet pile section in order to create box sections for girders 6 to 9. It has a mortar-like texture with high viscosity, which made it ideal for bonding the coarse aggregates to the flat surface of the GFRP sections, also discussed in Chapter 3.
4.2.2.6 **Coarse aggregates** – The silica stones used in bond system M2 were 4 to 9mm (0.16 to 0.35 in.) in diameter, the same as used in the slabs of Chapter 3.

4.2.2.7 **Mechanical shear stud connectors** – The steel threaded rod used was 12.7mm diameter B7 steel with yield and ultimate tensile strengths of 685MPa and 883MPa, respectively.

4.2.3 **Fabrication of Test Specimens**

GFRP plates and sheet pile sections were cut to size and their surfaces were cleaned thoroughly using acetone. For Type 3 girders 6 to 9, the first step in the construction process was to fabricate the box girder sections by adhering the flat GFRP sheets to the top of the sheet pile sections. This was done by applying a layer of the Sikadur30 mortar-like adhesive to the bond surfaces and lightly clamping the two units together. All sections (sheet pile sections, or built up box girders) were then fitted with the designated bond system. For the wet adhesive bond, the surface was coated with a thin layer of epoxy resin using a conventional paint brush (Figure 4-15), no more than 45 minutes prior to casting the concrete on the GFRP section while the epoxy was still wet. For the bonded coarse aggregates, a thick layer of the mortar-like adhesive was applied to the surface of the GFRP plates with a carefully controlled thickness, and then a layer of the silica aggregates was applied and pressed into the adhesive (Figure 4-16). Once the adhesive was fully hardened, any loose aggregates were brushed off. This left a rough surface that could interlock with the wet concrete. For the specimens fitted with mechanical shear connectors, holes were drilled through the GFRP sections. Shear studs were created by inserting the threaded steel rods into the holes, which were then fastened.
to the section using jam nuts and large washers on both sides of the plate. The nuts were then tightened to the manufacturer’s specified allowable torque (60Nm), in order to maximize friction between the nuts and the GFRP plates. The studs were arranged along two rows (one on each upper flange of the sheet pile sections) with a 100mm longitudinal spacing. This spacing was designed such that flexural failure could be achieved prior to bearing failure at the stud/GFRP interface. Unfortunately, in girder 3 this flexural failure was preceded by a vertical pullout of these non-headed studs (Figure 4-17(a)) as will be discussed later. Therefore, headed studs were used in the rest of the specimens (Figure 4-17(b)). All studs were installed such that they protruded 50mm from the upper surface of the GFRP section (Figure 4-17). It should be noted that, although girder 9 did not feature any concrete, it was still fitted with shear studs (M3) in order to enhance the connection between the sheet pile and the flat GFRP sheet.

After preparation of the GFRP sections, simple wooden wall systems (Figures 4-18) were erected around the perimeter of each girder in order to retain the concrete on top of the GFRP stay-in-place forms, and to form the sides of the concrete slabs. The void in girders 1 to 3 was created by adhering foam blocks to the GFRP formwork prior to pouring the concrete (Figure 4-18). A steel welded wire mesh with 3mm diameter wires spaced at 76mm was installed near the upper surface of the slab of each specimen, mainly for shrinkage crack control. Concrete was then cast (Figure 4-19) and the surface was finished and moist-cured for at least 7 days.

4.2.4 Test Setup and Instrumentation

Each specimen was tested using a symmetric 4-point bending setup with a span of 3100mm, and a 400mm long constant moment zone between the two loading points. A
schematic of this setup is shown in Figures 4-3 to 4-5 and 4-7 to 4-10, and photographs of the setup are shown in Figures 4-20 and 4-21. Load was applied at a rate of 1mm per minute using a 1000kN Reihle machine with an integrated load cell. Strains were monitored using electrical resistance strain gages and 100mm displacement-type strain transducers located within the constant moment zone of each specimen. Linear potentiometers (LPs) were used to monitor midspan deflections, as well as horizontal slip between the concrete and the GFRP forms. Full dimensioned schematics of all of the instrumentation used are shown in Figures 4-22 through 4-27. It should be noted, however, that not all of the data collected from these instruments is presented or discussed, as much of the instrumentation was used merely for redundancy, to validate data, and to insure linearity of strain profiles along the depth. Girders 3 and 9 incorporated 45 degree strain rosettes within one of their shear spans at the mid-height of one of their webs. This rosette was located at a distance from the nearest applied loading point equal to half of the section’s depth. Girder 8 incorporated additional strain gauges at the extreme compression fiber and a 100mm displacement-type strain transducer at the extreme tension fiber within one of the shear spans, 150mm from the nearest loading point. At this location, there were both longitudinal and transverse strain gages on the compression flange of the sheet pile section. The purpose of this additional instrumentation was to capture the strains, local buckling behaviour, and strain profile in girder 8 near the location of maximum bending moments without being affected by the solid grout diaphragm that was located beneath the loading points and within the constant moment zone of this particular specimen.
4.3 Results of the Experimental Program

A summary of test results from all specimens, including ultimate load, load at first slip, and failure modes, is provided in Table 4-2. The load-deflection plots for all specimens are compared in Figure 4-28. Figures 4-29 to 4-34 show the load-deflection responses for different groups to examine the effects of each parameter. Similarly, Figures 4-37 to 4-44 show the moment-curvature responses, Figures 4-47 to 4-49 show the load-slip responses, and Figures 4-50 to 4-54 show the load-axial strain responses. Failure modes are shown in Figures 4-57 to 4-61.

4.3.1 Effect of Bond Mechanism

4.3.1.1 Effect on Stiffness

Figure 4-29 shows the load-deflection curves for girders 1, 2, and 3 with wet concrete to wet epoxy bond (M1), bonded aggregates (M2), and non-headed steel studs combined with wet epoxy (M1 and M3), respectively. Also, figure 4-37 shows the bending moment-curvature response of the same specimens. It is evident from these plots that, prior to failure, the stiffness of all specimens is quite similar. This is logical because, as shown in Figure 4-47, none of these specimens exhibited any horizontal bond slip at the interface between the concrete and the GFRP sheet pile prior to failure; therefore, these specimens behaved monolithically prior to the sudden failure of their bond mechanisms. The specimens have the same cross-sectional configurations, and therefore behaved similarly. Although girder 3 utilized bond mechanism M3, which is a mechanical connection with discretely spaced interlocking elements, it also incorporated bond mechanism M1. Therefore, all three of these specimens incorporated bond mechanisms that generate a uniformly distributed shear connection between the concrete
and the GFRP section. As a result of this uniform shear connection, tension stiffening behaviour after cracking of the concrete is exceptional in all three specimens. This is evident by the smooth curved transition between the pre-cracked stiffness and post-cracked stiffness of these specimens, as seen in Figure 4-37. However, it should be noted that the presence of a void in the tension region of the concrete diminishes the effects of tension stiffening and reduces the cracking load, which is in any case small relative to the ultimate load. This will be discussed in more detail later. Although girder 3 incorporated both bond mechanisms M1 and M3, it is clear from Figure 4-29 that the behaviour of girders 1 and 3 was nearly identical below the failure load of girder 1. Beyond this load, girder 3 exhibited a slight reduction in stiffness. This was likely caused by a failure of its M1 bond mechanism first (the failure mode of girder 1), leaving only mechanism M3 responsible for resisting horizontal shear at the concrete / GFRP interface. It is logical that mechanism M3 alone would result in a slightly reduced stiffness because it is dependent upon mechanical interlocking shear studs. Between these studs, it is possible for horizontal shear slip to occur, which means that composite action is compromised and shear-induced deflections can occur. In fact, it is likely that the majority of the reduced stiffness occurred as a result of shear deflection since the reduced stiffness of girder 3 is only evident in its load-deflection plot (Figure 4-29) but not in the moment-curvature plot (Figure 4-37).

Figure 4-30 shows the load-deflection curves for girders 6 and 7, which have bond mechanisms (M1) and (M1 combined with M4), respectively. Figure 4-49 shows load versus slip at the interface between the concrete and the GFRP box girder for the two specimens, and Figure 4-38 shows their bending moment-curvature plots. It is evident from these plots that, prior to failure, the stiffnesses of girders 6 and 7 were quite similar,
which is similar to girders 1, 2, and 3. Again, this is logical since girders 6 and 7 both incorporated bond mechanism M1, and neither of these specimens exhibited significant slip at the interface between the concrete and the GFRP sheet pile prior to failure. It should be mentioned that girder 7 did exhibit some slip after failure of its M1 bond mechanism, thus mobilizing its M4 bond mechanism throughout the remainder of the test. This small amount of slip was punctuated by a very minor reduction in stiffness. It is apparent that in girders 3 and 7, their respective mechanical interlock bond mechanisms (M3 and M4, respectively) were not mobilized until failure of their adhesive bond mechanisms (M1) occurred. This can be explained by the fact that some slip must occur to mobilize mechanical shear connectors. If girders 3 and 7 did not incorporate bond mechanism M1, then it would have been likely that significant slip, and hence reduced stiffness, would take place. This was shown in slab girders 3 and 4 in Chapter 3.

4.3.1.2 Effect on Strength and Failure Mode

The ultimate strength of concrete members having FRP structural forms is highly dependent on the quality of bond between the two materials. It can be seen in Figures 4-29 and 4-30 that the ultimate strength of girders 1, 2, and 3, as well as girders 6 and 7 differed substantially. By carefully examining the interface surfaces of girders 1, 2, and 6 after failure, it was concluded that bond failure occurred within a very thin layer of the cement mortar, as in the slabs of Chapter 3. This was made evident by a visible layer of cement paste bonded to the GFRP sections (Figure 3-6(c)). Therefore, it can be said that the shear strength of bond mechanisms M1 and M2 are both dependent upon the shear strength of the cement paste. A typical value reported in literature for the shear strength of unconfined conventional cement mortar is 1MPa (Hegemier et al, 1978, and Pluijm, Pluijm,
It has also been shown in Chapter 3 that this value may be used to accurately model the performance of bond mechanism M1.

It is clear from Figure 4-29 that specimens incorporating mechanisms M1 and M2 behaved quite similarly. Figure 4-57 shows the typical bond failure of these specimens. Girder 3 exhibited a progressive failure that was punctuated by an initial debonding failure of mechanism M1, which appeared to occur at a similar load to the failure of girder 1. This initial failure is made evident by the reduction in stiffness discussed earlier. The ultimate failure of girder 3 occurred at a slightly higher load when mechanism M3 failed by pullout of the studs from the concrete slab near one of the supports (Figure 4-58). This was due to the absence of heads on the shear studs in mechanism M3. Therefore, the failure load of girder 3, although higher than those of girders 1 and 2, is considered somewhat premature. Girder 6 failed by debonding in a similar manner to girders 1 and 2, as shown in Figure 4-59. It was also noted that bond failure occurred between the GFRP flat plate and the sheet pile section. The adhesive at this interface has higher tensile and shear strengths than that used above the plate to bond to the concrete, yet a bond failure also occurred at this level. This may have been a secondary failure after debonding at the concrete/GFRP interface. Unlike girder 6, girder 7, which incorporated bond mechanisms M1 and M4 with headed studs, did not ultimately fail by pullout of the shear studs. It exhibited a slight reduction in stiffness (Figures 4-30 and 4-38) and slight horizontal shear slip (Figure 4-49) at a load similar to the failure load of girders 1 and 6, which indicates that mechanism M1 probably failed at this load. Its mechanism M4, however, did not fail at all. At approximately 430kN, a thin flat layer of concrete within one of the shear spans of girder 7 delaminated along a horizontal plane that intersects the top surfaces of the shear studs. It is noted that the
shear studs displace concrete, thereby creating a plane of weakness. This delamination caused a drop in load (Figure 4-30). The load then climbed back up to approximately 430kN, at which point the concrete slab crushed adjacent to one of the loading points. This crushing was immediately followed by buckling of the upper flange of the GFRP box section. Figure 4-60 shows the failure of girder 7.

By examining the load-longitudinal strain responses of girders 1, 2, and 3 in Figure 4-50 and girders 6 and 7 in Figure 4-51, it can be seen that the maximum measured longitudinal tensile strains in the GFRP sheet pile section at failure were well below the manufacturer’s specified ultimate tensile failure strain ($\varepsilon_{fr}$) from coupons of 0.0134 for girders 1, 2, and 3, and 0.0206 for girders 6 and 7. As such, tension failure was never immanent in any of these specimens. At the compression side, except for girder 7, the longitudinal compressive strain was well below 0.0035, specified by CAN/CSA-A23.3-94 for concrete crushing ($\varepsilon_{cc}$). Clearly girder 7 has exceeded this strain, and indeed compression failure has occurred.

4.3.2 Effect of Geometric Configuration

4.3.2.1 Effect on Stiffness

Figures 4-31 and 4-39 compare the load-deflection and moment-curvature responses of girders 1, 4, and 6 of three different cross-sectional configurations, but similar bond systems. It should be noted, however, that the total depth of all sections was identical; the area of concrete in compression for all specimens was quite similar; and the flat GFRP sheet that made up the top flange of the box girder of girder 6 was in close proximity to the neutral axis of the section. It is evident from these plots that the post-cracking stiffness of these specimens was nearly identical. However, the initial parts of
the curves near cracking were different. Because girder 4 incorporated far more concrete in its tension region than either of the other two specimens, it had the greatest cracking moment and exhibited a relatively abrupt change in stiffness after cracking (Figure 4-39). Girder 6 had no concrete in its tension region; hence it did not exhibit any cracking or pronounced changes in flexural stiffness. Although girders 1 and 4 incorporated the same bond mechanism, the flexural response of girder 4 is indicative of a lower tension stiffening coefficient ($\alpha_1$) than girder 1 (Figure 4-39). Since the tension stiffening coefficient of a reinforced concrete member is only dependent upon the quality of the bond between the concrete and the reinforcement, it is clear that not all of the concrete in the tension region of girder 4 contributed to tension stiffening. Girder 4 had a relatively large volume of concrete in its tension region that was not adjacent to its GFRP form reinforcement when compared with girder 1. This geometry was likely responsible for the discrepancy in their tension stiffening behaviours. For conventional steel reinforced concrete members, it has been shown that tension stiffening is only effective within a limited distance ($d_{TS}$) from the centroid of the reinforcing bars. This distance is generally accepted as fluctuating between 0 and 7.5 times the diameter of the reinforcing bars, thus having an average value of approximately 3.75 times the diameter of the reinforcing bars (Collins and Mitchell, 1997). The value of $d_{TS}$ is likely to differ for GFRP reinforcement due to its lower elastic modulus; however, it is clear from comparing the flexural responses of girders 1 and 4 that tension stiffening in concrete members reinforced by GFRP formwork is undoubtedly effective only within a limited distance from the form. While girder 4 had a higher cracking moment than girder 1 due to its greater cross-sectional area of concrete, girder 4 exhibited a more abrupt drop in stiffness after cracking occurred because a relatively small percentage of its concrete was able to contribute to
tension stiffening. This phenomenon generated the appearance of inferior tension stiffening behaviour in girder 4.

Figure 4-40 compares the moment-curvature responses of girders 3 and 7 of two different cross-sectional configurations, but with similar bond systems. The flexural response of these specimens compare in a manner that is similar to the comparison of girders 1 and 6. In general, the primary difference in behaviour between the slab-on-box sections (6 and 7) and all the other specimens is their lack of a pronounced cracking moment when loaded in positive bending.

Figures 4-35 and 4-45 compares the load-deflection and moment-curvature responses of girders 7 and 9 in positive bending. It is important to note that, due to local warping and buckling of the upper compression flange of girder 9, non-linear reversing strains were recorded by SG8 at the extreme compression fibre (Figure 4-55). Therefore, the moment-curvature plot for girder 9 can only be used to assess stiffness at bending moments below approximately 65kNm. Within this range, it was found that the flexural stiffness (EI) of girder 7 was approximately 75% greater than that of girder 9. This discrepancy can be attributed to the presence of a concrete slab on girder 7, which serves to increase the effective depth of the section, thus increasing its moment of inertia considerably. In fact, girder 9 began to soften considerably as local buckling of the upper flange began, as shown in the load-deflection response of the specimen (Figure 4-35) where the curve begins to flatten above approximately 190kN.

4.3.2.2 Effect on Strength

Although girders 1, 4, and 6 all exhibited failure by debonding of their M1 bond mechanisms, girders 4 and 6 resisted relatively higher loads than girder 1. This can likely
be attributed to the differences in bond surface area and configuration between the concrete and the GFRP form of girders 4 and 6, when compared with girder 1. The bond area of girder 1 was compromised by the presence of a void within the tension region of the concrete. Bond failure in girders 1 and 4 was likely initiated along the plane of the upper surface of the upper flange of the sheet pile section since this is the region in closest proximity to the neutral axis of the section, and hence the maximum shear region. It is clear that the total shear flow necessary to ensure composite action between this sheet pile and the concrete overlay is distributed over a larger bond area in girder 4 than in girder 1. Girder 6 also had a slightly smaller bond area than girder 4, but its bond area was relatively flat and regular. It is clear from Figure 4-52 that longitudinal tensile and compressive strains at failure were well below the expected ultimate failure values of the materials.

It is worth noting that the webs of the GFRP sections in girders 1 and 4 essentially behaved as stirrups, helping the composite hybrid sections to resist shear by resisting tension while the concrete located between the webs resisted diagonal compression. Therefore, the bond areas located on the diagonal GFRP webs of girders 1 and 4 were burdened with vertical shear stresses and tensile stresses in addition to horizontal shear stresses. This helps to explain why girder 6 was able to out-perform girder 1 despite their somewhat similar bond areas. From a strength-to-weight ratio stand point, it is quite clear that girder 6 out-performed girders 1 and 4 by 130% and 200%, respectively, while girder 1 outperformed girder 4 by 30%.

As of girders 7 and 9 in positive bending, it is clear from Figure 4-35 that the ultimate strength of girder 7 was approximately 86% higher than that of girder 9. This, once again, can be attributed to the presence of a concrete slab on girder 7, which served
to restrain the upper flange of the GFRP box section in order to prevent local buckling and at the same time resist higher compression forces. The slab also increased the effective depth and moment of inertia of the section. It is interesting to note that girder 9 failed in a manner that is reminiscent of a secondary failure mode evident in girder 7. In girder 9, the flange buckled in a vertical plane along one of the edges of the sheet pile section (Figure 4-63). This vertical displacement of the upper flange, in turn, caused severe deformation of the web. Similarly, once the concrete slab on girder 7 crushed, the upper flange of the GFRP box section was no longer restrained; therefore, immediately after this crushing occurred, the upper GFRP flange buckled in a manner that was nearly identical to the failure observed in girder 9 (Figure 4-60).

4.3.3 Performance under Negative Bending Conditions

4.3.3.1 General Behaviour

In continuous beams, the magnitude of negative bending moments often equals or exceeds that of positive bending moments. In this section, the configurations of girders 4 and 7, which were studied in positive bending, are also assessed in negative bending through girders 5 and 8, respectively. Figures 4-32 and 4-41 show the load-deflection and moment-curvature responses of these two girders. It is important to note that girder 8 had a 500mm long solid diaphragm within the constant moment zone, which would be a necessary detail over the interior supports of a continuous beam in order to prevent local crippling of the webs. For this reason, the mid-span flexural response of girder 8 was measured both within the constant moment zone and within one of the shear spans where the box section was left hollow, as shown in Figure 4-42. The flexural stiffness of girder 8 in its hollow shear span region was slightly lower than that of girder 5, which was likely
due to the presence of concrete in the compression region of girder 5 (Figures 4-32 and 4-41).

The failure mode of girder 5 was by lateral opening of the GFRP sheet pile section (Figure 4-61). This occurred as a result of large curvature in the specimen, which generated a net upward force component in the bottom flange and a net downward force component in the upper flange. This in turn caused the angled webs to flatten, thus laterally thrusting the tension flanges out to the sides and peeling the webs away from the concrete core. This phenomenon is similar in concept to ovalization exhibited by hollow round tubes (Ibrahim, 2000). Girder 8 was restrained from opening in this fashion due to the presence of the flat GFRP sheet acting as a tie between the two tension flanges of the sheet pile section. However, despite the resistance of girder 8 to this lateral opening of the GFRP section, its performance was not significantly better than that of girder 5. This was due to the fact that the GFRP box section was hollow outside of the constant moment zone, which allowed local buckling to occur in the webs and compression flanges of the GFRP box section (Figure 4-62), and ultimately lead to a compression failure of the GFRP section by a combination of crushing and local buckling. This buckling is evident by the abrupt shift in strains near failure as shown in Figures 4-53 and 4-54. The compressive strain at which this local buckling occurred was far below the manufacturer’s specified compressive failure strain of 0.0132. It is evident from Figure 4-42 that the flexural stiffness of girder 8 was significantly greater in the constant moment region where it incorporated a grout diaphragm within the compression zone, than within the shear span (just outside of the constant moment zone) where the box section was left hollow.
Figure 4-36 compares the load-deflection responses of girders 8 and 9 in negative bending. It is evident from this plot that the stiffness of these two specimens is quite similar; however, girder 8 is slightly stiffer. This may be partially attributed to the contribution of the concrete slab in tension prior to cracking, as well as the presence of a steel wire mesh within the concrete slab which acted as tensile reinforcement. Another reason for the slight discrepancy in the stiffness of these two specimens may have been the presence of a grout diaphragm in the constant moment zone of girder 8. It was mentioned earlier that this diaphragm caused a slight increase in flexural stiffness within the constant moment zone. Although girder 9 was only loaded to 80kN in negative bending, it is clear from its load-strain response (Figure 4-56) that local buckling had already begun in the compression flange of the sheet pile section. This was made evident by the abrupt change in the curve of the compression strain readings from SG1 just prior to terminating the test.

4.3.3.2 Comparison of Behaviour in Negative and Positive Moments

Figures 4-33 and 4-43 compare the load-deflection and moment-curvature plots of girders 4 and 5. It is evident from these plots that the flexural strength and stiffness (\(EI_{xx}\)) of the completely concrete filled section are approximately 45% and 175% higher in positive bending than in negative bending. Similarly, Figures 4-34 and 4-44 compare the load-deflection and moment-curvature plots of girders 7 and 8, respectively. The flexural strength and stiffness of this box configuration are approximately 90% and 225% higher in positive bending than in negative bending.

When a uniformly distributed load is applied to multi-span continuous beams of constant flexural stiffness, negative bending moments over the interior supports can often
equal or exceed the maximum positive bending moments found between supports. If flexural stiffness in negative bending is lower than in positive bending, however, the inflection points tend to be located closer to the supports than they would be in a beam of constant stiffness. This, in turn, reduces negative bending moments over the supports and increases positive bending moments between supports. Therefore, while the negative bending strength of girders 5 and 8 are substantially lower than their positive bending counterparts (girders 4 and 7, respectively), the fact that their negative bending flexural stiffness are also lower dictates that these sections could potentially be designed and optimized for continuous beam applications such that negative and positive flexural failures would occur almost simultaneously. This simultaneous failure mode would be ideal because it means that the section is fully utilized in all regions (mid-span and over supports) of the continuous beam, but not having much allowance for moment redistribution.

4.4 Comparison with Conventional Reinforced Concrete and Prestressed Sections

In this section, the moment-curvature responses of two girders studied in this chapter, namely girders 6 and 7, are compared with convention steel-reinforced concrete (RC) and steel-prestressed girders of the same concrete strength, geometry, and outer dimensions. Girder 6 failed in bond while girder 7 failed in flexure. The objectives were to establish the quantity and placement of reinforcing steel or prestressing tendons necessary to achieve the same moment at ultimate, and the same flexural stiffness as girders 6 and 7. Conventional cracked section analysis has been used to obtain the moment-curvature responses of the RC girders and the prestressed girders. This analysis was performed using computer software called Response2000, which was developed by
Evan Bentz and Professor Michael P. Collins at the University of Toronto. In RC sections, Steel rebar of 400MPa and 600MPa yield and ultimate strengths, respectively, has been used in this analysis. In prestressed members, low-relaxation grade 1860MPa steel 7-wire strands were used for the prestressing steel. For consistency, all prestressing steel was assumed to have an effective prestress force such that the stress in the strands was equal to $0.6f_u$ (1116MPa), which corresponds to a prestrain of 0.00558. Provisions of CAN/CSA-A23.3-94 regarding concrete cover and limitations on rebar and tendon spacing have been satisfied. As such, a minimum of 40mm clear cover was used over all bars and strands.

4.4.1 Comparison with Girder 6

For comparison with the bending moment at failure in girder 6 that failed in bond, a hypothetical RC section called RC6a was created. In order to achieve the same flexural strength as girder 6, it was necessary to use a row of 3-25M bars located 58mm (on centre) above the bottom of the member, and another row of 2-10M bars located 111mm (on centre) above the bottom of the member. The resultant member had a reinforcement ratio of 1.07%. Similarly, to achieve similar flexural stiffness of girder 6, hypothetical RC section RC6b required the use of a row of 3-15M bars located 48mm (on centre) above the bottom of the member. The resultant member had a reinforcement ratio of 0.36%. Figure 4-64(a and b) illustrates the design and geometry of these hypothetical sections, and Figure 4-66 compares the moment-curvature responses of RC6a and RC6b with the response of girder 6. It is evident from this plot that the RC member matching the strength of girder 6 had a stiffness that was approximately 90% higher than girder 6,
while the RC member matching the stiffness of girder 6 had a strength that was less than half that of girder 6.

Also, for comparison with the bending moment at failure in girder 6, a hypothetical prestressed section called PS6a was created. In order to achieve the same flexural strength as girder 6, it was necessary to use a row of 4-S13 strands located 47mm (on centre) above the bottom of the member. The design and geometry of this section is shown in Figure 4-65(a). The moment-curvature response of this section is compared with that of girder 6 in Figure 4-67. It is evident from this figure that section PS6a was considerably stiffer than girder 6 because of the very high cracking moment. In fact, the flexural stiffness of section PS6a was approximately 300% higher than that of girder 6. It was not possible to design a prestressed member with a comparable flexural stiffness to that of girder 6. This is because in a prestressed member cracking occurs at a relatively high load, and all or most of the concrete section contributes to flexural stiffness prior to cracking. The absence of concrete in the tension region of girder 6 dictates that it is softer than any prestressed section that can be designed using the assumed cross sectional geometry discussed earlier and shown in Figure 4-65(a). Therefore, unless an extremely low prestressing force is employed, which would essentially render the section as a non-prestressed reinforced concrete section anyway, a prestressed section with comparable external dimensions (non-voided) and comparable stiffness to girder 6 cannot be created.

4.4.2 Comparison with Girder 7

For comparison with the bending moment at failure in girder 7 that failed in flexure, a hypothetical RC section called RC7a was created. To achieve the same flexural strength, it was necessary to use a row of 3-30M located 65mm (on centre) above the
bottom of the member, and another row of 2-30M bars located 137mm (on centre) above the bottom of the member. The resultant member had a heavy reinforcement ratio of 2.49%. Similarly, for comparison with the flexural stiffness, hypothetical RC section RC7b required a row of 2-15M bars located 48mm (on centre) above the bottom of the member, and another row of 2-10M bars located 84mm (on centre) above the bottom of the member, to achieve the same stiffness. The resultant member had a reinforcement ratio of 0.37%. Figure 4-64(c and d) illustrates the design and geometry of these hypothetical sections. Figure 4-68 compares the moment-curvature response of RC7a and RC7b with the response of girder 7. It is evident from this plot that the RC member matching the strength of girder 7 had a stiffness that was approximately 185% higher than that of girder 7, while the RC member matching the stiffness of girder 7 had a flexural strength equal to less than a third of the strength of girder 6.

Also, for comparison with the bending moment at failure in girder 7, hypothetical prestressed section PS7a required the use of a row of 3-S15 strands located 48mm (on centre) above the bottom of the member, and a row of 2-S15 strands located 109mm (on centre) above the bottom of the member as shown in Figure 4-65(b). The moment-curvature response of this section is compared with that of girder 7 in Figure 4-69. It is evident from this figure that section PS7a was considerably stiffer than girder 7. In fact, the flexural stiffness of section PS7a was approximately 380% higher than that of girder 7. Again, it was not possible to design a prestressed member with comparable flexural stiffness to that of girder 7. The reason for this was discussed earlier.
Table 4-1. Girder specimen descriptions

<table>
<thead>
<tr>
<th>Girder #</th>
<th>Type</th>
<th>Bond Mechanisms</th>
<th>Concrete Fill</th>
<th>Bending Orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>M1</td>
<td>Voided Concrete</td>
<td>Positive</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>M2</td>
<td>Voided Concrete</td>
<td>Positive</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>M1 &amp; M3</td>
<td>Voided Concrete</td>
<td>Positive</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>M1</td>
<td>Completely Filled</td>
<td>Positive</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>M1</td>
<td>Completely Filled</td>
<td>Negative</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>M1</td>
<td>Slab on Box Girder</td>
<td>Positive</td>
</tr>
<tr>
<td>7</td>
<td>3</td>
<td>M1 &amp; M4</td>
<td>Slab on Box Girder</td>
<td>Positive</td>
</tr>
<tr>
<td>8</td>
<td>3</td>
<td>M1 &amp; M4</td>
<td>Slab on Box Girder</td>
<td>Negative</td>
</tr>
<tr>
<td>9</td>
<td>3</td>
<td>M3</td>
<td>None</td>
<td>Positive (a) &amp; Negative (b)</td>
</tr>
</tbody>
</table>

Table 4-2. Summary of girder test results

<table>
<thead>
<tr>
<th>Girder #</th>
<th>Peak Load (kN)</th>
<th>Load at Initial Bond Slip (kN)</th>
<th>Mode of Failure</th>
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<tr>
<td>1</td>
<td>250.0</td>
<td>250.0</td>
<td>Bond failure at concrete/GFRP interface</td>
</tr>
<tr>
<td>2</td>
<td>269.5</td>
<td>269.5</td>
<td>Bond failure at concrete/GFRP interface</td>
</tr>
<tr>
<td>3</td>
<td>317.0</td>
<td>317.0</td>
<td>Vertical pull-out of non-headed steel shear studs</td>
</tr>
<tr>
<td>4</td>
<td>294.0</td>
<td>294.0</td>
<td>Bond failure at concrete/GFRP interface</td>
</tr>
<tr>
<td>5</td>
<td>204.2</td>
<td>204.2</td>
<td>Lateral opening of GFRP sheet pile section</td>
</tr>
<tr>
<td>6</td>
<td>285.0</td>
<td>244.7</td>
<td>Bond failure at concrete/GFRP interface</td>
</tr>
<tr>
<td>7</td>
<td>430.0</td>
<td>230.5</td>
<td>Delamination and crushing of concrete slab</td>
</tr>
<tr>
<td>8</td>
<td>227.0</td>
<td>47.0</td>
<td>Local buckling and crushing of GFRP section</td>
</tr>
<tr>
<td>9a</td>
<td>230.7</td>
<td>NA</td>
<td>Buckling of upper compression flange of GFRP box section</td>
</tr>
</tbody>
</table>
Figure 4-1(a). Data pertaining to GFRP sheet pile section used to fabricate girder specimens 1 to 5 (adapted from Creative Pultrusions Inc web site www.creativepultrusions.com)
Figure 4-1(b). Data pertaining to GFRP sheet pile section used to fabricate girder specimens 6 to 9 (adapted from Creative Pultrusions Inc web site www.creativepultrusions.com)
## MATERIAL PROPERTIES

**Pultex**® Fiber Reinforced Polymer Flat Sheets

**Metric Version**

1500 Series - Thermoset Polyester – Olive Green
1525 Series - Thermoset Polyester Class 1 FR – Slate Gray (Dark Gray)
1625 Series - Thermoset Vinyl Ester Class 1 FR – Beige

The following data was derived from ASTM coupon and full section testing. The results are average values based on random sampling and testing of production lots. Composite materials are not homogeneous and, therefore, the location of the coupon extraction can cause variances in the coupon test results. Creative Pultrusions publishes an average value of random samples from production lots.

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Test</th>
<th>Units</th>
<th>1500/1625 Series</th>
<th>1625 Series</th>
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<tr>
<td><strong>Mechanical</strong></td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>Flexural Strength, Flatwise (LW)</td>
<td>D700</td>
<td>MPa</td>
<td>240.6</td>
<td>240.6</td>
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<tr>
<td>Flexural Strength, Flatwise (CW)</td>
<td>D700</td>
<td>MPa</td>
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<td>103.1</td>
</tr>
<tr>
<td>Flexural Modulus, Flatwise (LW)</td>
<td>D790</td>
<td>GPa</td>
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<td>12.7</td>
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<tr>
<td>Flexural Modulus, Flatwise (CW)</td>
<td>D790</td>
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<tr>
<td>Tensile Strength (LW)</td>
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<td>MPa</td>
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<td>MPa</td>
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<td>68.7</td>
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<tr>
<td>Tensile Modulus (LW)</td>
<td>D638</td>
<td>GPa</td>
<td>12.4</td>
<td>12.4</td>
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<tr>
<td>Tensile Modulus (CW)</td>
<td>D638</td>
<td>GPa</td>
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<td>6.9</td>
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<tr>
<td>Compressive Strength, Edgewise (LW)</td>
<td>D695</td>
<td>MPa</td>
<td>165.0</td>
<td>165.0</td>
</tr>
<tr>
<td>Compressive Strength, Edgewise (CW)</td>
<td>D695</td>
<td>MPa</td>
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<td>110.0</td>
</tr>
<tr>
<td>Compressive Modulus, Edgewise (LW)</td>
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<td>GPa</td>
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<td>12.4</td>
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<tr>
<td>Compressive Modulus, Edgewise (CW)</td>
<td>D695</td>
<td>GPa</td>
<td>6.9</td>
<td>6.9</td>
</tr>
<tr>
<td>Notched Iod Impact (LW)</td>
<td>D256</td>
<td>J</td>
<td>1.0676</td>
<td>1.0676</td>
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<tr>
<td>Notched Iod Impact (CW)</td>
<td>D256</td>
<td>J</td>
<td>296.9</td>
<td>296.9</td>
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<tr>
<td>Bearing Strength (LW)</td>
<td>D953</td>
<td>MPa</td>
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<tr>
<td>Bearing Strength (CW)</td>
<td>D953</td>
<td>MPa</td>
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<td>220.0</td>
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<tr>
<td>Polson's Ratio (LW)</td>
<td>D3009</td>
<td>mm/mm</td>
<td>0.32</td>
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<tr>
<td>Polson's Ratio (CW)</td>
<td>D3009</td>
<td>mm/mm</td>
<td>0.25</td>
<td>0.25</td>
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**Physical**

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<th>1625 Series</th>
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<tr>
<td>Brinell Hardness (10)</td>
<td>D383</td>
<td></td>
<td>40</td>
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<td>Water Absorption</td>
<td>D570</td>
<td>% Max</td>
<td>0.5</td>
<td>0.5</td>
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<tr>
<td>Density</td>
<td>D792</td>
<td>g/cm³</td>
<td>1.561.93</td>
<td>1.561.93</td>
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<td>Specific Gravity</td>
<td>D792</td>
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<td>1.95</td>
<td>1.95</td>
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<tr>
<td>Coefficient of Thermal Expansion (LW)</td>
<td>D896</td>
<td>10°<code>-6K</code>-1</td>
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**Electrical**

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<th>ASTM Test</th>
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<th>1500/1625 Series</th>
<th>1625 Series</th>
</tr>
</thead>
<tbody>
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<td>Arc Resistance (LW)</td>
<td>D495</td>
<td>seconds</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Dielectric Strength (LW)</td>
<td>D149</td>
<td>kV/mm</td>
<td>1.58</td>
<td>1.58</td>
</tr>
<tr>
<td>Dielectric Strength (FE)</td>
<td>D149</td>
<td>kV/mm</td>
<td>7.9</td>
<td>7.9</td>
</tr>
<tr>
<td>Dielectric Constant (FE)</td>
<td>D150</td>
<td>pF/µF</td>
<td>5.2</td>
<td>5.2</td>
</tr>
</tbody>
</table>

---

1. *Not a typical surface test that reflects the Brinell Hardness, but does not indicate hardness.

Additional Properties located on back

**CREATIVE PULTRUSIONS, INC.**

24 Industrial Lane |大胆Bend, CA 95001 | 800-839-4456 | Fax: 831-839-4774
Web site: [http://www.creativepultrusions.com](http://www.creativepultrusions.com) | E-mail: cpi@putrude.com

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Figure 4-1(c). Data pertaining to GFRP flat sheet section used to fabricate box girders (adapted from Creative Pultrusions Inc web site www.creativepultrusions.com)
Figure 4-2. GFRP sections used to fabricate test specimens
Figure 4-3. Dimensions and test setup for girders 1 to 3

- **Geometry**
  - 610mm
  - 245mm
  - 150mm
  - Creative Pultrusions SuperLoc 1610

- **Construction**
  1. EpoxySystems 7SC adhesive for bonding to wet concrete on all surfaces of concrete/GFRP interface
  2. Silica pebble aggregates bonded to horizontal surfaces of concrete/GFRP interface using Sika Sikadur30
  3. EpoxySystems 7SC adhesive for bonding to wet concrete on all surfaces of concrete/GFRP interface, and non-headed shear studs on upper flanges

- Details:
  - 76x76x3 mm steel mesh
  - Symmetric 4-point uniaxial bending
  - 250mm
  - 400mm
  - 3100mm
  - void
  - diaphragm
Figure 4-4. Dimensions and test setup for girder 4

Geometry

Construction

EpoxySystems 7SC adhesive for bonding to wet concrete on all surfaces of concrete/GFRP interface
Figure 4-5. Dimensions and test setup for girder 5

- **Geometry**
  - 610mm width
  - 70mm height
  - 76x76x3 mm steel mesh

- **Construction**
  - EpoxySystems 7SC adhesive for bonding to wet concrete on all surfaces of concrete/GFRP interface
  - Symmetric 4-point uniaxial bending
  - 3100mm length
  - 400mm span
  - Steel mesh

Simulates region over interior support of continuous beam
Figure 4-6. Picture of girder 4
Figure 4-7. Dimensions and test setup for girders 6 and 7
Figure 4-8. Dimensions and test setup for girder 8

Creative Pultrusions
Pultex 1500 flat sheet
(9.5mm thick)

Creative Pultrusions
SuperLoc 1610

Geometry

Construction

EpoxySystems 7SC adhesive
for bonding to wet concrete on all surfaces of concrete/GFRP interface, and headed shear studs on upper flanges

Sika Sikadur30 Epoxy at interface between sheet pile section and flat GFRP sheet

Simulates region over interior support of continuous beam

76x76x3 mm steel mesh

Symmetric 4-point uniaxial bending

hollow box

500mm diaphragm

hollow box

76x76x3 mm steel mesh

3100mm
Figure 4-9. Dimensions and test setup for positive bending of girder 9.
Figure 4-10. Dimensions and test setup for negative bending of girder 9
Figure 4-11. Cross-section of girders 6, 7, and 8

Figure 4-12. Coupons used for assessing material properties of GFRP sections
Figure 4-13. Stress-strain curves from coupon tension tests used to assess material properties of GFRP sheet piles.

(a) GFRP sheet pile sections from girders 1, 2, 3, 4, and 5

(b) GFRP sheet pile sections from girders 6, 7, 8, and 9

\[ f_c = 465 \text{ MPa} \]
\[ f_u = 561 \text{ MPa} \]
\[ f_c = 398.3 \text{ MPa} \]
\[ f_u = 336.6 \text{ MPa} \]
\[ f_c = 574 \text{ MPa} \]
Figure 4-14. Stress-strain curves from coupon tension tests used to assess material properties of GFRP plates

Figure 4-15. Application of M1 adhesive bond mechanism
Figure 4-16. M2 silica pebble aggregate bond mechanism

Figure 4-17. Headed and non-headed studs (bond mechanisms M3 and M4)
Figure 4-18. Foam used to create void within Type 1 girder specimens

Figure 4-19. Concrete pour and vibration
Figure 4-20. Positive bending test setup

Figure 4-21. Negative bending test setup
All gauges mounted at mid-span (centre of constant moment zone)

Figure 4-22. Strain gauges at mid-span
Figure 4-23. Strain gauges and rosettes within shear span
Figure 4-24. PI gauges at mid-span

All gauges mounted at mid-span (centre of constant moment zone)

Shaded boxes indicate inclusion of instrument

Constant moment zone
Figure 4-25. PI gauges within shear span
All gauges mounted at mid-span (centre of constant moment zone)

Figure 4-26. Linear Potentiometers (LPs) at mid-span
Shear span

An identical setup was used to measure bond slip at the concrete/GFRP interface at both ends of each girder.

Shaded boxes indicate inclusion of instrument.

**Figure 4-27. Linear Potentiometers (LPs) at ends of girder**
Figure 4-28. Load-deflection responses of all girder specimens
Figure 4-29. Load-deflection responses of girder specimens 1, 2, and 3

Figure 4-30. Load-deflection responses of girder specimens 6 and 7
Figure 4-31. Load-deflection responses of girder specimens 1, 4, and 6

Figure 4-32. Load-deflection responses of girder specimens 5 and 8
Figure 4-33. Load-deflection responses of girder specimens 4 and 5

Figure 4-34. Load-deflection responses of girder specimens 7 and 8
Figure 4-35. Load-deflection responses of girder specimens 7 and 9a in positive bending.

Figure 4-36. Load-deflection responses of girder specimens 8 and 9b in negative bending.
Figure 4-37. Moment-curvature response of girder specimens 1, 2, and 3

Figure 4-38. Moment-curvature response of girder specimens 6 and 7
Figure 4-39. Moment-curvature response of girder specimens 1, 4, and 6

Figure 4-40. Moment-curvature response of girder specimens 6 and 7
Figure 4-41. Moment-curvature response of girder specimens 5 and 8

Figure 4-42. Moment-curvature response of girder specimen 8 within diaphragm region (constant moment zone), and within hollow box region (shear span)

Note: Response for specimen 8 was taken within hollow box region of the girder (shear span)
Figure 4-43. Moment-curvature responses of girder specimens 4 and 5

Figure 4-44. Moment-curvature responses of girder specimens 7 and 8

Note: Response for specimen 8 was taken within hollow box region of the girder (shear span)
Figure 4-45. Moment-curvature responses of girder specimens 9a and 7

Note: Response for specimen 8 was taken within hollow box region of the girder (shear span)

Figure 4-46. Moment-curvature responses of girder specimens 9b and 8
Figure 4-47. Load versus bond slip of girder specimens 1, 2, and 3

Figure 4-48. Load versus bond slip of girder specimen 4
Figure 4-49. Load versus bond slip of girder specimens 6 and 7
Figure 4-50. Load-strain responses of girder specimens 1, 2, and 3

Figure 4-51. Load-strain responses of girder specimens 6 and 7
Figure 4-52. Load-strain responses of girder specimens 1, 4, and 6

Figure 4-53. Load-strain responses of girder specimens 5 and 8

Note: Strains for specimen 8 were taken from slightly outside of constant moment zone.
Figure 4-54. Load-strain responses of girder specimen 8 illustrating the effect of the diaphragm within the constant moment zone.
Figure 4-55. Load-strain responses of girder specimens 9a and 7

Figure 4-56. Load-strain responses of girder specimens 9b and 8
Figure 4-57. Bond failure in girders 1, 2, and 4

Figure 4-58. Pull-out of mechanism M3 studs in girder 3
Figure 4-59. Bond failure in girder 6

Figure 4-60. Delamination and crushing of concrete in girder 7
Figure 4-61. Lateral opening of sheet pile in girder 5

Figure 4-62. Buckling and crushing of webs and compression flange of girder 8
Figure 4-63. Warping and local buckling of upper flange of girder 9 in positive bending (9a)
Figure 4-64. Hypothetical conventional RC sections designed to match strength and stiffness of girders 6 and 7

(a) RC6a designed to match strength of girder 6

(b) RC6b designed to match stiffness of girder 6

(c) RC7a designed to match strength of girder 7

(d) RC7b designed to match stiffness of girder 7

Figure 4-64. Hypothetical conventional RC sections designed to match strength and stiffness of girders 6 and 7
Figure 4-65. Hypothetical conventional prestressed concrete sections designed to match the strength of girders 6 and 7

(a) PS6a designed to match strength of girder 6

(b) PS7a designed to match strength of girder 7

\[ f_e = 0.6 f_u \]
Figure 4-66. Moment-curvature response of RC6a, RC6b, and girder 6

Figure 4-67. Moment-curvature response of PS6a and girder 6
Figure 4-68. Moment-curvature response of RC7a, RC7b, and girder 7

Figure 4-69. Moment-curvature response of PS7a and girder 7
CHAPTER 5: ANALYTICAL MODEL OF GFRP STAY-IN-PLACE SYSTEM FOR CONCRETE GIRDERS

5.1 Introduction

Chapter 4 has introduced the experimental program in which nine girders, referred to as 1 to 9, were fabricated using trapezoidal pultruded GFRP sheet pile sections as stay-in-place structural formwork for concrete. All girders had a total length of 3350mm, and incorporated the GFRP sheet pile section shown in Figure 4-1(a and b). Girders of three unique configurations were tested, namely, sections completely filled with concrete, sections with a voided concrete core, and an all-GFRP box girders with a top concrete slab. Four different bond mechanisms between the concrete and the GFRP section were investigated.

In this chapter, an analytical model has been developed to predict the complete flexural behavior of the specimens. The model is intended for modeling specimens tested in positive bending only (i.e. girders 1, 2, 3, 4, 6, and 7). The model is somewhat similar to that described for the slabs in Chapter 3 in that it establishes the moment-curvature response of the section, which is then terminated at a point governed either by flexure or bond failure. Conventional cracked section analysis based on equilibrium of forces and strain compatibility is carried out, assuming full bond between the concrete and the GFRP section prior to failure. A layer-by-layer technique is adopted in order to integrate stresses in the concrete and the GFRP throughout the depth of the member.
5.2 Description of the Analytical Model

5.2.1 Constitutive Relationships

The analytical model was based upon the following constitutive relationships for concrete and GFRP:

5.2.1.1 Concrete in Compression:

For concrete in compression, the widely accepted stress-strain model presented by Collins and Mitchell (1997) was adopted. As shown in Figure 5-1, the stress \( f_c \) relates to strain \( \varepsilon_c \) in accordance with the following equations:

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

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

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

where, \( f'_c \) is the compressive strength of concrete, and \( \varepsilon'_c \) is the strain in concrete at a stress of \( f'_c \). When the strain \( \varepsilon_c \) exceeds 0.0035 (CAN/CSA-A23.3-94) it is assumed that concrete has crushed and therefore adopts a zero stress.

5.2.1.2 Concrete in Tension:

For concrete in tension, a linear stress-strain relationship was assumed prior to cracking.

\[
f_c = E_c \varepsilon_c \quad \text{for} \quad f_c \leq f'_c
\]
Initial modulus of elasticity for concrete $E_c$ was approximated in accordance with a suggested formula in CAN/CSA-A23.3-94 as follows.

$$E_c = \left(3300\sqrt{f'_{c'}} + 6900\left(\frac{\gamma_c}{2300}\right)^{1.5}\right)$$

(5-3)

where $\gamma_c$ is the density of concrete. It should be noted that the equation used to calculate $E_c$ represents the secant modulus at 0.4$f'_{c'}$ stress, and therefore is not representative of the initial tangent modulus at zero strain, $E_{co}$; however, for most cases, the difference between these values is negligible (Collins and Mitchell, 1997). Therefore, for the purposes of this model, $E_{co}$ was approximated using the aforementioned equation for $E_c$.

Modulus of rupture was taken as $f_r = 0.6\sqrt{f'_{c'}}$ (CAN/CSA-A23.3-94). When the rupture strain of concrete is exceeded, an average tensile stress in the concrete was assumed to be smeared throughout the cracked region using a tension stiffening function posed by Vecchio and Collins (Collins and Mitchell, 1997). This tension stiffening relationship is illustrated in Figure 5-1, and describes the stress versus strain relationship as follows:

$$f_c = \frac{\alpha_1\alpha_2 f_r}{1 + \sqrt{500(\varepsilon_c - \varepsilon_r)}} \quad \text{for } f_c > f_r$$

(5-4)

where $\varepsilon_r$ is the strain at cracking. $\alpha_1$ is a factor accounting for the bond characteristics of the reinforcement and is taken as 1.0 to model the zero-slip characteristics of adhesively bonded reinforcement as observed in the experiments (Chapter 4). $\alpha_2$ is a factor accounting for sustained or repeated loading, taken as 1.0 for short term quasi-static monotonic loading.

For conventional steel-reinforced concrete members, it has been shown that tension stiffening is only effective within a limited distance ($d_{TS}$) from the centroid of the reinforcing bars. This distance is generally accepted as fluctuating between 0 and 7.5
times the diameter of the reinforcing bars, thus having an average value of approximately 3.75 times the diameter of the reinforcing bars. The value of $d_{TS}$ is likely to differ for GFRP reinforcement due to its lower elastic modulus. In lieu of more extensive studies of tension stiffening behaviour of FRP-reinforced concrete, the model discussed in this investigation assumed the value of $d_{TS}$ to be equal to 3.75 times the thickness of the GFRP form. Figure 5-2 illustrates how this concept applies to Type 2 specimens (girder 4) in positive bending. Type 1 specimens (girders 1, 2, and 3) would have the same $d_{TS}$ value as Type 2 specimens, except that much of the cracked zero stress region shown in Figure 5-2 would be displaced by the void present in Type 1 specimens, thus altering the post-cracking behaviour of these specimens.

5.2.1.3 GFRP Sections in Tension or Compression:

The resin matrix properties, fibre properties, and stacking sequences for the pultruded GFRP sections used in this investigation are proprietary data. However, the manufacturer has published some data pertaining to the mechanical behaviour of these sections (Figure 4-1). Also, tension coupon tests were conducted by the author (Figures 4-12 to 4-14). A linear stress-strain relationship was assumed in both tension and compression. The elastic modulus, tensile strength, and compression strength of the GFRP sheet pile sections were taken as 26.2GPa, 276MPa, and 241MPa, respectively, when simulating Types 1 and 2 specimens (girders 1 to 5). The elastic modulus, tensile strength, and compression strength of the GFRP sheet pile sections were taken as 26.2MPa, 517MPa, and 345MPa, respectively, when simulating Type 3 specimens (girders 6 to 9). Although manufacturer’s data were used, Young’s modulus agreed well with coupon test results for the sheet pile sections. Since no tension or compression
failure occurred in the GFRP sections, the use of strengths reported by the manufacturer, which were lower than those from the coupons, should not matter. For the GFRP flat sheet sections, there was some discrepancy between the manufacturer’s published data and the data obtained from coupon tests performed by the author. An elastic modulus of 18GPa was ultimately selected in accordance with the author’s experimental flexural coupon tests. In general, the material properties of the flat GFRP sections are less important due to the close proximity of this material to the neutral axis of the section.

5.2.2 Section Geometry

In order to reduce the complexity of the model, the geometry of the GFRP sheet pile section was simplified to the form illustrated in Figure 5-3(a), from the actual section used experimentally, shown in Figure 4-1. The primary difference is the omission of the pin-and-eye joint details along the edges of the upper flanges. It was decided that this omission would have a negligible effect on the flexural response of the member due to the close proximity of this detail to the neutral axis of the section. All other aspects of the sections were modeled in a manner that precisely mimics the geometry of the actual sections. The assumed geometry of each of the specimen types is shown in Figures 5-3(b to d). Despite the simplified geometry, integrating stresses over the section remained quite complex. As mentioned earlier, the section was divided into several horizontal layers in order to account for the non-linear stress distribution. As such, it was necessary to calculate the average width of each material (concrete or GFRP) within each horizontal layer in the section, and subsequently multiply this width by the thickness of the layer in order to obtain the cross-sectional area of each material present in a layer. In order to do
so, the section was divided into 5 general regions (Figure 5-4), and each region was assessed in a unique fashion as follows:

Region 1:

Region 1 comprised all layers within the lower flange of the GFRP sheet pile section. In this region, only GFRP is present, and the width of the GFRP present in a layer within this region \( W_{GFRP} \) was calculated as follows:

\[
W_{GFRP} = B_{lf} + (2t(i + 0.5)\tan \theta)
\]  

\((5-5)\)

where \( t \) is the thickness of one discrete layer within the flange, \( i \) is the summation of all layers from the bottom of the flange to the layer of interest, \( B_{lf} \) is the width of the lower flange taken at the bottom of the section, and \( \theta \) is the angle of the webs of the sheet pile section measured with respect to the vertical. In lieu of this rigorous approach, one can possibly use a constant width, \( B_{lf} \), for all layers within this region.

Region 2:

Region 2 was defined as any layer at an elevation between the upper and lower flanges of the sheet pile section. In this region, the width of GFRP present in a given layer was calculated as follows:

\[
W_{GFRP} = \frac{2t_{web}}{\cos \theta}
\]  

\((5-6)\)

where \( t_{web} \) is the thickness of the web of the sheet pile section. The width of concrete present in a given layer in this region varied depending upon whether the section being modeled was of Type 1, 2, or 3. For Type 2 specimens (girders 4 and 5), the web was
totally filled with concrete. Therefore, for these specimens, the width of concrete present in a given layer \( W_{\text{concrete}} \) was calculated as follows:

\[
W_{\text{concrete}} = \left( B_y - \frac{2t_{\text{web}}}{\cos \theta} \right) + \left( 2t(i + 0.5)\tan \theta \right)
\]  

(5-7)

In Type 1 specimens (girders 1, 2, and 3), the width of a concrete layer could be calculated in an identical manner as described for Type 2 specimens (Equation 5-7) with the exception that the width of the void \( B_{\text{void}} \) had to be subtracted from the resultant value. In Type 3 specimens (girders 6, 7, and 8), there is no concrete within Region 2.

Region 3:

Region 3 comprised all layers at an elevation that passes through the upper flange of the GFRP sheet pile section. In this region, the width of GFRP present in a given layer was calculated as follows:

\[
W_{\text{GFRP}} = \left( B - B_{\text{wt}} + \frac{2t_{\text{web}}}{\cos \theta} \right) - \left( 2t(i + 0.5) - \left( H_{\text{pile}} - t_{\text{flange}} \right) \right) \right) \tan \theta
\]  

(5-8)

where \( B \) is the total width of the sheet pile section, \( H_{\text{pile}} \) is the total depth of the sheet pile section, \( t_{\text{flange}} \) is the thickness of the flanges of the sheet pile section (the same value for both upper and lower flanges), and \( B_{\text{wt}} \) is the distance between the outer surfaces of the webs of the sheet pile section at the elevation where the webs meet the bottom surface of the upper flange. One could also simplify this by assuming a constant width \( (B - B_{\text{wt}}) \) for all layers in this region. The width of concrete present in a given layer in this region was calculated as follows:

\[
W_{\text{concrete}} = \left( B_y - \frac{2t_{\text{web}}}{\cos \theta} \right) + \left( 2t(i + 0.5)\tan \theta \right)
\]  

(5-9)
Again, this could have been simplified as $W_{\text{concrete}} = B_{\text{wt}}$.

**Region 4:**

Region 4 comprised all layers at an elevation within the thickness of the flat GFRP plate that makes up the upper flange of the GFRP box section in Type 3 specimens (girders 6, 7, and 8). The width of GFRP in each layer in this region is simply equal to the total width of the section, $B$. In all other specimens (girders 1 though 5), there is no GFRP present in a layer within this region, and the width of concrete in each layer in this region is equal to the total width of the section, $B$.

**Region 5:**

Region 5 comprised all layers at an elevation above Region 4. In this region, all layers are composed entirely of concrete, and have a width equal to the total width of the section, $B$.

**5.2.3 Algorithm for Moment-Curvature Response**

Utilizing the aforementioned constitutive relationships for the materials and geometric relationships for the cross-section, a computer algorithm was written in FORTRAN90 to establish the moment-curvature response of the member, in a similar manner to that used for the slabs in Chapter 3, as follows:

**Step 1:** An initial strain is assumed at the extreme compression fiber of the section.
Step 2: The location of the neutral axis is then assumed in order to establish a complete strain profile assuming a linear strain distribution.

Step 3: The stresses in the concrete and the GFRP are then determined within a finite number (1000) of layers throughout the depth of the cross section using the aforementioned stress-strain relationships for concrete and GFRP (Figure 5-2).

Step 4: The stresses are integrated with the areas of their respective layers and summed in tension and compression. If the sum is not equal to approximately zero, then a new value of the neutral axis depth is assumed in step 2 and the process is repeated until equilibrium is satisfied.

Step 5: The sum of the moments generated by all forces from each layer ($M$) is calculated and the corresponding curvature ($\psi$) is also calculated as the slope of the strain profile for the particular compressive strain assumed at step 1. This provides one point on the $M-\psi$ response. The strain at the extreme tension fiber is then increased slightly in step 1 and the process is repeated, to establish the full $M-\psi$ response.

A simplified flowchart illustrating the procedure for establishing the moment-curvature response is provided in Figure 5-5.

5.2.4 Failure Criteria

Two failure criteria were considered in order to account for the possibility of flexural (tension or compression) failure, or bond failure. Concrete shear (diagonal
tension) failure was not considered to occur, given the relatively long span of this type of girder. In order to model these failure criteria, the aforementioned moment-curvature algorithm was repeated with the exclusion of tension stiffening in concrete to establish the termination (failure) point. The purpose of this omission was to account for the fact that both flexure and bond failure are likely to occur at the location of a crack. Therefore, the flexural response of each member is established first by an algorithm incorporating tension stiffening, but the termination point was established by a second algorithm at a crack location that assumes no tension stiffening. This second algorithm included a few additional elements necessary to check for the various failure criteria as discussed next.

Throughout the formulation of the moment-curvature plot, the maximum values of compression and tension strains are constantly monitored. A flexural failure is defined by an event in which either of these strains exceeds the failure strain of the concrete or the GFRP, respectively.

To check for bond failure, the horizontal shear stress ($\tau$) at any level in the section can be calculated using the following fundamental equation of mechanics:

$$
\tau = \frac{VQ_i}{I_{xx}B_i}
$$

(5-10)

where $V$ is the vertical shear force experienced by the girder at a given cross section (bending moment divided by shear span), $I_{xx}$ is the second moment of area of the section about its neutral axis, and $B_i$ is the width of the section at the elevation of interest. $Q_i$ is the first moment of area of the region above or below the elevation of interest, taken about the neutral axis of the cross-section of the hybrid section. It should be noted that
both $I_{xx}$ and $Q_i$ were transformed in accordance with the relative elastic moduli of the materials present in the sections (Figure 5-6), which will later be discussed in detail. In most cases, the maximum horizontal shear stress occurs at or near the neutral axis of a section. Since the neutral axis of these hybrid sections tends to be located slightly above the upper flange of the GFRP sheet pile section, it can be assumed that bond failure between the concrete and the GFRP section is initiated at the elevation of the upper surface of the top flange of the GFRP section. Prior to cracking, this is a fairly simplistic calculation. However, once flexural cracking occurs, the moment of inertia of the section is reduced and the neutral axis of the section begins to shift. Therefore, at any given load, the values of $I_{xx}$ and $Q_i$ are only constant within the uncracked region, where bending moment ($M$) is less than the cracking moment ($M_{cr}$); however, they vary substantially in the cracked zone. Within the cracked regions, $I_{xx}$ and $Q_i$ are calculated on the basis of a cracked section; hence, tension stiffening was not employed when checking for this failure criteria. This is to account for the fact that debonding (i.e. when $\tau_{max}$ is reached) is typically initiated at the location of a crack. $I_{xx}$ is calculated by summing the second moment of area of every intact layer within the section about the neutral axis of the section, and $Q_i$ is calculated by summing the first moment of area of every intact layer within the section about the neutral axis of the section.

It should be noted that, when calculating $I_{xx}$, and $Q_i$, and to account for the concrete non-linearity, the width of each layer in the section is transformed by the secant elastic modular ratio of the material present in that layer, which depends on the strain level in that layer, relative to the elastic modulus of unstrained concrete $E_{co}$, as shown in Figure 5-6. This is important since the elastic modulus of concrete effectively decreases
as strain is increased. For layers composed of GFRP, the elastic modulus ratio is simply calculated by dividing the elastic modulus of the GFRP section by $E_{co}$.

For each value of $M$ in the cracked section analysis, the values of $I_{xx}$, $V$, $Q_i$, and $B_i$ were used to calculate $\tau$ at the elevation of the upper surface of the top flange of the GFRP section, using Equation 5-10. This calculation was executed at the location of maximum shear and bending moment (immediately adjacent to a loading point). This is because shear is constant within the shear spans, and the location of maximum bending moment would have the lowest value of $I_{xx}$, and thus the highest bond shear stress. Debonding is assumed to have been initiated when the maximum $\tau$ value reaches the ultimate bond shear strength $\tau_{ult}$. After carefully examining the interface surfaces of girders 1, 2, and 6 after failure, it was concluded that bond failure occurred within a very thin layer of the cement mortar, and not within the adhesive bond line. This was made evident by a visible layer of cement paste bonded to the GFRP sections, which confirms the observation reported in Chapter 3. Therefore, it can be said that the shear strength of bond mechanisms M1 and M2 are both dependent upon the shear strength of the cement paste. A typical value reported in literature for the shear strength of unconfined conventional cement mortar is 1MPa (Hegemier et al, 1987, and Pluijm, 1993). Therefore, for the purposes of this model, it was assumed that the ultimate bond shear strength $\tau_{ult}$ may be taken as 1MPa. It should be noted that the progressive debonding (i.e. analysis beyond first debonding) discussed in Chapter 3 was ignored. It was shown in Chapter 3 that this progressive failure was only evident in relatively slender members. The relatively low shear-span to depth ratios used for the girders in this study dictated that failure by complete debonding occurred immediately after the bond shear stress

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reached the value of $\tau_{ult}$ at the location of maximum bending moment, thus mitigating the need to employ the complex progressive debonding analysis discussed in Chapter 3.

In members incorporating headed studs (M4), it was assumed that flexural failure would precede bond failure. As such, for these specimens, bond failure criterion was ignored and the model was allowed to continue until it was terminated by a flexural failure.

### 5.2.5 Load-Deflection Predictions

Once the moment-curvature response of a section is determined, it can then be used to establish the load-deflection relationship using basic mechanisms. In order to do so, an initial load is assumed, half of the span is divided into several segments (Figure 5-7), each with a length of $d_x$, and the average bending moment experienced within each of these regions ($M_i$) is calculated in accordance with the loading scheme being used (4-point bending in this case). The data from the previously established moment-curvature response is then employed in order to determine the average curvature in the member within each region ($\psi_i$). The product of the length of each region and the curvature value within that region ($\psi_i \times d_i$) gives the change in slope ($\Delta \theta_i$) of the member within that region. If it is assumed that the slope of the member at mid-span is zero, then the slope of the member at midpoint of each region ($\theta_i$) is equal to the summation of the values of $\Delta \theta_i$ for all regions between the mid-span and the point of interest. The product of the length of each region and the average slope within that region ($\theta_i \times d_i$) gives the change in elevation ($\Delta v_i$) of the member within that region. The summation of all of the $\Delta v_i$ values for each region between the mid-span and the support gives the total mid-span deflection.
of the member ($\delta$). This entire process is repeated at various load levels in order to establish the complete load-deflection response of the member.

### 5.3 Validation of Analytical Model

The aforementioned model was primarily aimed at predicting the behaviour of specimens incorporating either the M1 bond mechanism, or a combination of the M1 and M4 bond mechanisms. As such, the model was validated using the experimental results from girders 1, 4, 6, and 7. The experimental and analytical moment-curvature, load-deflection, and moment-strain responses of each of these specimens are compared in Figures 5-8 to 5-11, 5-12 to 5-15, and 5-16 to 5-19, respectively. In general, it was found that the moment-curvature responses and failure loads generated by the model had excellent agreement with the experimental results. The moment-curvature response is very sensitive to tension stiffening effects in the vicinity of the cracking moment; however, the parameters used ($a_1 = 1$, and $d_{TS} = 3.75$ times the thickness of the GFRP form) appear to have resulted in a response that is quite consistent with the experimental results. The model accurately predicts that all specimens incorporating adhesive bond mechanisms (M1 and M2) fail by debonding at the concrete/GFRP interface, and provides a good estimate of their failure loads, although it underestimated the failure load of girders 1 and 2 by 12.5 percent and 16.9 percent, respectively. It is clear from the plots that the load-deflection responses generated by the model tended to underestimate deflection by approximately 14 to 20 percent. This can likely be attributed to factors not accounted for in the model such as shear deflection.
5.4 Parametric Study

The analytical model was used to study the effects of five parameters on the performance of the girders being investigated. These parameters are: (a) compressive strength of the concrete used, (b) bond strength of the cement paste at the concrete/GFRP interface together with shear span-to-depth ratio, (c) angle of the webs of the GFRP sheet pile section, (d) thickness of the flanges of the GFRP sheet pile section, and (e) width of the concrete slab.

In order to properly assess each of these parameters, it was important that certain aspects of the section were set as constants. As such, all sections studied were similar to girder 4 in that they were simply composed of a single trapezoidal sheet pile section with a solid concrete fill, and an adhesive bond mechanism at the GFRP/concrete interface. The total depth of the sheet pile section was 254mm, the width of its lower flange was 208mm, the webs were 7.62mm thick, and the flanges were 8.89mm thick. The depth of concrete was set such that a 70mm thick concrete slab existed above the upper flange of the sheet pile section. The shear span of all specimens was assumed to be 1350mm, similar to experimental testing. All mechanical properties of the sheet pile section were assumed to be consistent with the Creative Pultrusions SuperLoc 1610 sections used in the experimental testing.

While each of the parameters was studied, all other parameters were kept constant. Therefore, each parameter was given a default value to be used when that parameter was not being studied. These default values were set as follows: compressive strength of concrete $f_c'$ was 35MPa, bond strength was 1MPa, webs were at an angle of 20 degrees from the vertical, the flanges of the sheet pile sections were 8.89mm thick, and the total
width of the section was 610mm. These values were essentially the same as those of the specimens that were experimentally tested.

5.4.1 Effect of Concrete Strength, $f_c'$

Concrete strength can sometimes be quite variable depending upon the quality control that is exercised. Therefore, it was important to quantify the sensitivity of this system to changing values of concrete compressive strength $f_c'$. Four concrete strengths were studied: 25MPa, 35MPa, 45MPa, and 55MPa. Figure 5-20(a) shows the moment-curvature plots for all cases, as well as the point at which each girder would exhibit debonding failure given the shear span used. As would be expected, flexural strength and stiffness both increase slightly as concrete strength is increased. This effect is most pronounced just prior to flexural failure. All four girders exhibited bond failure at approximately the same load, which occurred prior to flexural failure that would have otherwise occurred by crushing of concrete. Therefore, it can be said that, for the shear span and bond strength used, there exists a concrete strength that will allow flexural failure to occur prior to bond failure; however, as shown in Figure 5-20(b), this concrete strength would be quite low at approximately 18.5MPa.

5.4.2 Effect of Bond Strength and Shear Span-to-Depth Ratio

Perhaps one of the most critical parameters that should be studied for this structural system is the bond strength at the concrete/GFRP interface. The ability to precisely adjust such a parameter experimentally would be extremely difficult and impractical since bond failure tends to occur within a thin layer of cement mortar adjacent to the bond line, and not at the bond line itself. However, the analytical model described
in this report makes it possible to study this parameter so that the sensitivity of this structural system to bond strength can be quantified. Using the specified shear span of 1350mm, a series of specimens were analyzed with bond strength ($\tau_{ult}$) values ranging between 0.25MPa and 2.5MPa. This range covers the scatter of values of bond strength of mortar shown in Figure 2-7 in the literature.

The relationship between bond strength and bending moment at failure is plotted in Figure 5-21(a), while the relationship between bond strength and failure load is plotted in Figure 5-21(b). For a shear span of 1350mm, the plot shows a constant failure load of 320kN when bond strengths lower than approximately 1.1MPa are used. Within this range, the strength of the member is governed by the flexural strength of the GFRP sheet pile section alone assuming no composite action with the concrete core and ignoring local buckling. When bond strength exceeds 1.1MPa, a linear trend of increasing failure load results until bond strength reaches approximately 1.35MPa. Within this range, bond failure is the governing mode of failure. When bond strength is further increased beyond 1.35MPa, flexural failure of the hybrid member (i.e. concrete and GFRP with full composite action) precedes bond failure; hence, failure load is constant within this region of the plot as well. It is interesting to note that during experimental testing, girder 4 (the specimen that most closely resembles the girder currently being discussed) had a bond strength of 1MPa and a shear span of 1350mm, and yet it did not reach the 320kN load stipulated by Figure 5-21(b). This was because the experimental test was terminated immediately following bond failure; however, it should be noted that the 320kN failure load estimate is based upon flexural tension and compression failure, and not local buckling which is likely to occur first.
Because this plot was only applicable to specimens with the specific geometry and loading configuration used for the specimens analyzed, some addition plots were made using specimens with different shear spans ranging from 1350mm to 2000mm (i.e. shear span-to-total depth ratio of 4.17 to 6.17). It is clear that, as shear span is increased, it is possible to induce a flexural failure with lower bond strength values. Likewise, for the default 1MPa bond strength used in this study, there exists a minimum shear span of slightly less than 2000mm that would enable flexural failure to precede bond failure. In Figure 5-21(a), it is evident that, regardless of their shear spans, all specimens are limited to a maximum possible bending moment at failure that is a property of the cross-sectional geometry. As shear span is increased, the bond strength necessary to achieve this maximum bending moment is reduced; hence, flexural failure of the composite member tends to govern more in cases of longer shear spans.

5.4.3 Effect of Angle of Webs of Sheet Pile Section

Although the pultruded sheet pile sections used in this investigation are mass produced with specific dimensions, the pultrusion process enables the production of sections with a wide range of geometries and dimensions. As such, the effects of changing the cross-sectional geometry of the sheet pile section have been studied. One of the manners in which this geometry could be varied would be by varying the angle of the webs of the sheet pile section measured with respect to the vertical. This angle was varied while maintaining the width of the bottom flange constant at 208mm. The standard SuperLoc 1610 section used in the experimental investigation had webs that were at an angle of 20 degrees from the vertical. In this parametric study, four web angles were studied, namely, zero degrees (vertical), 10 degrees, 20 degrees, and 30
degrees. Figure 5-22(a) shows the moment-curvature plots and the point at which debonding would occur for each of these specimens. This plot illustrates that altering the angle of the webs has a negligible effect on the flexural response of these specimens, including both bond failure and flexural failure moments (Figure 5-22(b)). There is a very slight increase in stiffness as the angle of the webs is increased. It is logical that, with the relatively small angles used in this study, a negligible effect was found since the width of the bottom flange was kept constant. Greater angles could not be employed in this parametric study due to the limited width of the top flange of the sheet pile section.

**5.4.4 Effect of Thickness of Flanges of Sheet Pile Section**

Another manner in which the geometry of the sheet pile section could be altered is by changing the thickness of the flanges of the section. Four flange thicknesses were studied, namely, 4mm, 10mm, 16mm, and 22mm. Figure 5-23(a) shows the moment-curvature plots, as well as the point at which debonding would occur for each of these specimens. As expected, it is clear from this plot that the flexural strength and stiffness of the specimens improve significantly as the thickness of the flanges is increased. It is also evident that, regardless of flange thickness, all specimens exhibited debonding at similar loads. As illustrated in Figure 5-23(b), with the given shear span (1350mm) and bond strength (1MPa), it would not be possible to adjust the thickness of the flanges such that flexural failure occurs prior to debonding at the concrete/GFRP interface.

It is interesting to note that the tension stiffening behaviour of these specimens in the vicinity of cracking moment differ substantially from one another. While specimens with thin flanges showed an abrupt drop in bending moment immediately after cracking, specimens with relatively thick flanges exhibited a smooth transition between pre-
cracking stiffness and post-cracking stiffness. This is indicative of the fact that the effects of tension stiffening are only pronounced in regions that are in close proximity to reinforcement (the sheet pile section). As discussed earlier, tension stiffening occurs within a limited distance \( (d_{TS}) \), which is a function of the thickness of the GFRP flange, from the reinforcement or formwork. Therefore, as the thickness of the lower flange of the sheet pile section was increased, the proportion of the concrete below the neutral axis that was affected by tension stiffening was increased; hence, the specimen with the greatest flange thickness (22mm) showed the strongest tension stiffening performance. It is also worth noting that the pre-cracking stiffness of all specimens was nearly identical. This is due to the fact that the GFRP in the sheet pile section has a very similar modulus of elasticity to concrete. Therefore, although more GFRP was present in the flanges of some specimens than others, this GFRP merely displaced concrete of similar stiffness.

5.4.5 Effect of Width of Concrete Slab

The idea of varying the width of the concrete slab implies that the total width of the GFRP sheet pile section should also be varied. Therefore, while studying this parameter, it was assumed that the geometry of the lower flange and webs of the sheet pile sections were maintained at their default dimensions, while only the breadth of the upper flanges was varied. One of the implications of such an assumption is that the bonded area between concrete and the sheet pile section increases as the width of the slab increases. Four slab widths were studied, namely, 500mm, 600mm, 700mm, and 800mm. Figure 5-24(a) shows the moment-curvature responses and the point at which debonding occurs for each of the girders. Since the flexural strength and stiffness of these specimens increased as the width of slab was increased, an additional plot (Figure 5-24(b)) was
made in which the moments were normalized by dividing by slab widths. Since these hybrid sections are intended to be placed adjacent to each other as a broad one-way deck, this normalization serves to provide a flexural response per unit width of decking. In general, the flexural strength and stiffness of these specimens was found to be inversely proportional to the width of the concrete slab. Conversely, for a given shear span (1350mm in this case), the load at which bond failure occurred in these specimens increased as the width of the slabs was increased. Therefore, it can be said that, for a given shear span and bond strength (1MPa in this case), there exists an optimum width of section that will enable a flexural failure prior to debonding at the concrete/GFRP interface. As shown in Figure 5-24(c), for the 1350mm shear spans used in this investigation, this optimum width would be slightly greater than 1500mm. It should be noted that in creating Figure 5-24(c), three additional data points (1300mm, 1400mm, and 1500mm) were used in order to ensure the accuracy of the trend-line used to extrapolate out to the optimum slab width.
Figure 5-1. Stress-strain relationship for concrete in uniaxial compression and tension

Collins and Mitchell (1997)

Uniaxial Compression

Uniaxial Tension

Stress, $f_c$

Strain, $\varepsilon_c$

$\varepsilon_{c'} = 0.0035$

$0.4f'_c$

Tension stiffening

Collins and Mitchell (1997)
Figure 5-2. Stress distribution in Type 2 specimens in positive bending (girder 4)

- Fully intact concrete in compression and uncracked tension zone
- Cracked concrete with zero stress
- Cracked concrete with stress due to tension stiffening
Figure 5-3 Assumed geometry of different sections.
Figure 5-4. Discrete regions in cross-sectional geometry
Figure 5-5. Flow chart illustrating algorithm used for determining moment-curvature response
Concrete stress within uncracked regions and dTS region

Concrete with zero stress when cracked (Figure 5-2)

Actual geometry of cross-section

Concrete stress within uncracked regions and dTS region

GFRP Stress

Transformed geometry of cross-section

Layer i

Layer k

For Concrete:

Transformed width in \(i\) = Actual width in \(i\) \(\times \frac{E_{ci}}{E_c}\)

Transformed width in \(k\) = Actual width in \(k\) \(\times \frac{E_{ck}}{E_c}\)

For FRP:

Transformed width in any layer = Actual width in layer \(\times \frac{E_f}{E_c}\)

Figure 5-6. Section transformation accounting for variable elastic modulus in concrete
Figure 5-7. Schematic of the process for determining load-deflection response
Figure 5-8. Experimental versus predicted moment-curvature responses of girder specimen 1

Figure 5-9. Experimental versus predicted moment-curvature responses of girder specimen 4
Figure 5-10. Experimental versus predicted moment-curvature responses of girder specimen 6

Figure 5-11. Experimental versus predicted moment-curvature responses of girder specimen 7
Figure 5-12. Experimental versus predicted load-deflection responses of girder specimen 1

Figure 5-13. Experimental versus predicted load-deflection responses of girder specimen 4
Figure 5-14. Experimental versus predicted load-deflection responses of girder specimen 6

Figure 5-15. Experimental versus predicted load-deflection responses of girder specimen 7
Figure 5-16. Moment-strain response of girder specimen 1

Figure 5-17. Moment-strain response of girder specimen 4
Figure 5-18. Moment-strain response of girder specimen 6

Figure 5-19. Moment-strain response of girder specimen 7
Figure 5-20. Effect of concrete strength ($f_c'$) on:

(a) Moment-curvature responses

(b) Variation of debonding to flexural moment ratio with $f_c'$

Figure 5-20. Effect of concrete strength ($f_c'$)
(a) Variation of ultimate bending moment and bond strength

(b) Variation of failure load and bond strength

Figure 5-21. Effect of bond strength ($\tau_{ult}$) when various shear spans are used
Figure 5-22. Effect of web angle ($\theta$)

(a) Moment-curvature response

(b) Variation of debonding to flexural moment ratio with web angle
Figure 5-23. Effect of flange thickness ($t$)

(a) Moment-curvature response

(b) Affect of flange thickness on moment ratio

Figure 5-23. Effect of flange thickness ($t$)
Figure 5-24. Effect of slab width ($B$)
### Figure 5-24. Effect of slab width ($B$) (continued)

#### (c) Variation of moment ratio with slab width

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**Figure 5-24. Effect of slab width ($B$) (continued)**
CHAPTER 6: SUMMARY AND CONCLUSIONS

6.1 Introduction

The concept of stay-in-place structural forms simplifies and accelerates construction to a great extent. Indeed, it promises reduced road closures and traffic delays, in the case of bridge applications, which is a serious problem for the economy and a major inconvenience for the public. Commercially available pultruded GFRP sections have excellent potential as stay-in-place structural open formwork for concrete flexural members. The GFRP forms can fully or partially replace internal steel rebar, thereby mitigating the corrosion problems that threaten most conventional steel reinforced concrete structures. The inherent location of GFRP open forms, at the bottom of the member, makes these members well suited to applications in which positive bending is the predominant mechanism engaging the structure. Nevertheless, with some modifications, continuity in girders can also be achieved.

6.2 Summary and Conclusions

6.2.1 One-Way Slabs with Flat Pultruded GFRP Sheets

Eight concrete slabs were fabricated using pultruded flat GFRP sheets, and were tested in four-point uniaxial monotonic bending. Three bond mechanisms were explored in order to achieve composite action between the concrete and the GFRP plates. The effect of varying the GFRP reinforcement ratio was assessed using specimens of various concrete thicknesses. An analytical model was developed to predict the full behavior and strength of these hybrid slabs. A key feature of the model is its capability to detect flexural, shear or bond failures. Also, the performance of the hybrid slabs was assessed by
comparing them to conventional steel-reinforced concrete slabs of comparable external dimensions. The following conclusions were drawn:

1. Using readily available GFRP plates as structural forms for concrete slabs is quite feasible. The most effective bond system, in terms of structural performance and ease and speed of fabrication, was the wet adhesive bonding of fresh concrete to the GFRP plate.

2. The adhesive bonding methods led to a significantly higher stiffness, than that of the mechanically bonded slabs using shear connectors. This is attributed to a complete bond throughout loading in the former, until failure occurred suddenly by debonding; whereas in the latter, progressive slip between the GFRP plate and concrete occurred.

3. Failure was typically due to debonding of the GFRP plate, after flexural or shear cracking in the concrete. The measured low longitudinal strains at mid span suggest that tension failure of the GFRP plate is very unlikely in this system. Compression failure of concrete was imminent in some of the slabs.

4. While flexural stiffness of the slabs increased as their reinforcement ratio was increased, the effect of reinforcement ratio on ultimate moment could not be assessed because of the consistent bond failure, rather than the flexural failure.

5. Conventional steel-reinforced concrete slabs of comparable external dimensions are severely hindered by the requirements to satisfy the limitations of concrete cover. As a result, the concrete-GFRP hybrid system is superior in that the reinforcement is located at the extreme tension fibre. Also, substantial amounts of
steel reinforcement are required to match the flexural strength and stiffness of the hybrid sections.

6. Steel-reinforced concrete slabs of comparable flexural strength were considerably stiffer than their concrete-GFRP hybrid counterparts, whereas steel-reinforced concrete slabs of comparable stiffness had considerably lower flexural strength.

7. The behaviour of the concrete-GFRP hybrid slabs studied in this investigation can be modeled quite accurately using conventional cracked-section analysis techniques, in conjunction with a multi-stepped failure criterion approach that covers flexural, shear and bond failures.

8. Adhesive bond failure consistently occurred within a very thin layer of cement mortar of the concrete slab. Therefore, the use of the typical shear strength of 1MPa for cement mortar, established from research on masonry, to model bond failure was quite successful.

6.2.2 Girders using Trapezoidal GFRP Sheet Pile Sections: Experimental Investigation

Nine girders were fabricated using pultruded trapezoidal GFRP sheet pile sections and tested in four-point monotonic bending. Four bond mechanisms were explored in order to examine composite action. Three different cross-section configurations were explored, including the effect of implementing a void within the concrete fill, and the use of a built-up all-GFRP box girder system with a thin concrete compression flange. The system was tested in both positive and negative bending to simulate continuity. The following conclusions were drawn:
1. Similar to the slabs, girders incorporating either adhesive to wet concrete bond or bonded aggregates systems failed by debonding at the concrete/GFRP interface, within a thin layer of cement paste. A thin layer of mortar remained adhered to the GFRP section after failure. The concrete compressive strain and GFRP tensile strain were well below their ultimate values.

2. The specimen incorporating headed shear studs combined with the adhesive bond system was able to achieve flexural failure by concrete crushing, which occurred at a 45% higher load than that reached by specimens incorporating only the adhesive bond system.

3. The specimen incorporating non-headed studs in addition to the adhesive bond system achieved only a modest increase, 24% in strength, over the specimens incorporating only the adhesive bond system. In this case, failure occurred when the studs pulled out prematurely from the concrete slab.

4. Similar to the slabs, in all adhesively bonded and bonded aggregate girders, excellent monolithic composite action was observed prior to the sudden debonding failure. This was evident by the consistent lack of slip between the concrete and the GFRP throughout the loading history.

5. Including a void in the tension region of the concrete allowed for reduced self weight, without a significant reduction in stiffness. However, the reduced bonded area at the interface due to the void being extended to the GFRP bottom flange resulted in a small reduction, 13%, in failure load.

6. The built-up all-GFRP box section with a concrete slab showed superior performance. When it included headed studs in addition to adhesive bonding, it showed 35% to 70% higher strength than any other specimen. This also occurred
in conjunction with a 50% and 65% reduction in weight relative to the voided core and the totally filled specimens, respectively. When the box girder included adhesive bonding only, it showed similar strength to the adhesively bonded totally filled specimen, and 15% higher strength than the adhesively bonded voided core specimen.

7. The flexural strength of the girders is higher in positive bending than in negative bending by 45% to 90%, depending on cross-sectional configuration. However, since the stiffness is also higher in positive bending by 175% to 225%, the system lends itself to an efficient use in continuous beams, where it can be fully utilized in both positive and negative bending regions.

8. In negative bending, premature failure of the totally filled section occurred by lateral opening of the GFRP sheet pile. In actual applications, however, the sections would be placed side-by-side so that they restrain each other laterally. In the built-up box sections, the flat plate served as a tie restraining this opening action. In this case, local buckling of the GFRP sheet pile occurred just outside of the grout-filled diaphragm under the load.

6.2.3 Girders Using Trapezoidal GFRP Sheet Pile Sections: Analytical Investigation

An analytical model was developed to predict the flexural responses and failure loads of the girders reported in Chapter 4 of this investigation. The model employed conventional cracked section analysis in conjunction with a multi-stepped failure criteria check that accounted for flexural and bond failures. The model showed very good agreement with test results. It was then used in a parametric study to investigate the effects of compressive strength of concrete, bond strength at the interface, angle of the
webs of the sheet pile section, thickness of the flanges, and width of the concrete slab, on
the flexural stiffness, strength, and failure modes. The following conclusions were
drawn:

1. The flexural behaviour and strength of the concrete/GFRP hybrid girders studied
   in this investigation can be modeled quite accurately using conventional cracked-
   section analysis techniques in conjunction with a multi-stepped failure criterion
   approach that covers flexural and bond failures. Bond failure criteria based on the
typical shear strength of 1MPa of masonry mortar joints is reasonably accurate.

2. For concrete flexural members reinforced by GFRP stay-in-place open structural
   forms and employing a strong adhesive bonding mechanism at the concrete/GFRP
   interface, it is reasonable to assume a tension stiffening coefficient (\(\alpha_I\)) of 1; and
   an effective tension stiffening zone thickness (\(d_{TS}\)) of 3.75 times the thickness of
   the form.

3. In most practical applications where shear span-to-depth ratios are far greater than
   the 4.17 used in this investigation, flexural failure will likely precede bond failure
   despite the bond strength being limited to 1MPa.

4. Increasing the compressive strength of concrete increases flexural strength and
   stiffness slightly, but does not have a significant effect on the load at which bond
   failure occurs.

5. Debonding failure load is quite sensitive to the value of bond strength. Using
   shear strength values of cement mortar ranging from 0.25 to 2.5MPa resulted in
   failure loads that varied by over 25%. It is important to note, however, that the
conventional value established for masonry mortar joints of 1MPa is quite reasonable for design purposes.

6. Increasing the thickness of the flanges of the sheet pile section has a significant effect on increasing flexural strength and stiffness, as well as improving tension stiffening behaviour. However, it has a negligible effect on the debonding failure load.

7. For a given shear span and bond strength, there exists an optimum width of concrete slab that would enable flexural failure to precede bond failure. However, the flexural stiffness per unit width of deck tends to be highest when considerably narrower concrete slabs are used.

8. Adjusting the angle of the webs of the sheet pile section, while maintaining a constant width of tension flange, has an insignificant effect on flexural behaviour.

### 6.3 Recommendations for Future Work

This study has illustrated that the use of GFRP sections as stay-in-place structural open formwork for concrete flexural members is quite promising, and indeed this technology should be researched further. It is important that the following aspects are studied prior to implementation of the technologies discussed in this report:

1. The fatigue performance of concrete/GFRP hybrid members should be assessed under both high cycle and low cycle loading conditions. Special attention should be given to the fatigue life of the bond at the concrete/GFRP interface.

2. Environmental effects should be thoroughly studied for structures employing the aforementioned technologies. It is possible that differential thermal expansion could affect bond at the concrete/GFRP interface. Freeze-thaw effects should be
studied as the bond may be susceptible to icing and subsequent frost heave of moisture within cracks in the concrete near the concrete/GFRP interface.

3. In order to more accurately model the stiffness of large concrete/GFRP hybrid members of this type, it will be necessary to conduct further research on the tension stiffening behaviour of concrete members reinforced with externally bonded GFRP sections.

While this work has shown that the aforementioned structural systems perform quite well, the study also highlighted some aspects of the technology that require further refinement, as well as some other potential applications of the technology that may be developed. Key topics requiring further work are listed below:

1. Currently, all adhesive based bond mechanisms between the concrete and the GFRP were limited by the shear strength of the cement mortar (1.0MPa) layer in contact with the bond line. It may be advantageous to develop an adhesive based bond system that is somehow capable of generating a shear connection between the concrete and GFRP that exceeds the shear strength of cement mortar. Perhaps a more hydrophilic adhesive system could be used such that it can better infiltrate the cement mortar, thus potentially engaging the aggregate interlock.

2. This study was focused only on concrete/GFRP hybrid members in uniaxial bending. It is recommended that this technology be further developed for applications that generate biaxial bending conditions. Two-way concrete slabs or bridge decks reinforced with GFRP forms may prove to better utilize the membrane characteristics of the GFRP sheets.
6.4 Practical Applications

Assuming bond strength of 1MPa, dictated by the shear strength of cement mortar, it was shown that any completely filled (Type 2) hybrid girder with a shear span greater than 1750mm will be governed by flexural capacity and not by bond strength or diagonal tension shear strength. As such, there are two primary aspects of design that must be considered when employing Type 2 girders in actual structural applications: flexural strength, and deflection requirements. Designing for ultimate limit states would simply be a matter of assessing the factored load requirements in accordance with the appropriate building code, and limiting the span such that the highest factored bending moment in the span does not exceed the ultimate flexural capacity of the member stipulated by the analytical model described in Chapter 5. In order to design for serviceability limit states, it would be necessary to establish the flexural stiffness of the member \( (EI_{xx}) \). This value can be taken as the slope of the moment-curvature response of the Type 2 girder section as determined experimentally (Chapter 4) or analytically (Chapter 5). The value of \( EI_{xx} \) can then be used to calculate the maximum possible span that can be used such that, under the required service loads, deflections do not exceed the allowable deflections stipulated by the appropriate building code.
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APPENDIX 1: DETAILED DESCRIPTION OF SCIENTIFIC METHOD

Instrumentation and Precision of Measurements

1. **Data Acquisition**: All data channels were monitored and recorded using a personal computer based digital data acquisition system (Vishay StrainSmart System 5000).

2. **Strain Gauges**: All strain measurements applicable to GFRP surfaces in this investigation were made using electrical resistance foil gauges with a 5mm gauge length (Showa, type N11-FA-5-120-11).

3. **Linear Potentiometers (LPs)**: All linear displacement measurements in this investigation were made using linear potentiometers with a 100mm stroke (NovoTechnik, type TRS 100).

4. **Displacement-Type Strain Transducers (PI Gauges)**: All strain measurements applicable to concrete surface in this investigation were made using displacement-type strain transducers (PI gauges) with a 100mm gauge length and a maximum range of ±5mm (Tokyo Sokki Kenkyuo Co., type PI-5-100).

5. **Application and Measurement of Loads**: Loads were applied to specimens using a Reihle screw-driven loading machine. Loads were applied using stroke control at a constant rate of 1mm per minute in order to ensure quasi-static loading conditions. The loading machine incorporated an integrated load cell with an assignable load range of up to ±900kN. For the purposes of this investigation, a load range of ±500kN was employed in order to maximize measurement precision.
while ensuring that the load ranged enveloped the maximum expected load resisted by the specimens.

**Comments on Precision of Specimen Fabrication**

1. **Dimensions:** All specimens were constructed using pultruded GFRP sections as stay-in-place structural open formwork. As such, the dimensions of these GFRP sections may be considered to be accurate to within 0.01mm as per the manufacturer’s published data sheets. In the case of the flat GFRP plates, the width of the plates was cut using a hand-held circular saw; therefore, the width of these sheets is accurate to within approximately 3mm (thickness of the saw blade). Measurements for all additional wooden wall forms were made using a conventional retractable measuring tape with 1mm graduations, and markings were made using a permanent marker with a line thickness of approximately 3mm. Unfortunately, uncontrollable factors such as warping of the wooden forms caused imprecision in the dimensions of the formwork. For the most part, the resulting dimensional discrepancies were no greater than 5mm; however, it was found that the width of the members sometimes varied by as much as 10mm near mid-span. The depth of concrete in the members was very well controlled to within approximately 4mm of the design depth; as such the total depth of all specimens was fairly consistent with the specified dimensions discussed in chapters 3 and 4.

2. **Materials:** For each specimen fabricated, three concrete cylinders were fabricated using the same batch of concrete with a diameter of 4” and a height of 8”.  

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Similarly, 3 grout cylinders were fabricated for each specimen incorporating grout diaphragms. At the time of testing each specimen, its cylinders were tested in uniaxial compression in accordance with ASTM C39. The slab specimens shared the same batch of concrete as used for Type 1 girder specimens (specimens 1, 2, and 3). The results of the concrete tests are summarized in Tables A1 and A2. For each type of GFRP section, 3 dog-bone style coupons were fabricated and tested in accordance with ASTM D3039/D3039M. The results of these coupon tests are summarized in Table A3. The stress-strain responses of these coupons are shown in Figure 4-13.

Table A1. Concrete cylinder strength data

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<td><strong>4.7</strong></td>
<td><strong>1.4</strong></td>
</tr>
</tbody>
</table>

<table>
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<tr>
<th>Sheet Piles for specimens 1 to 5</th>
<th>Tensile Strength (MPa)</th>
<th>Elastic Modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon 1</td>
<td>359.7</td>
<td>NA</td>
</tr>
<tr>
<td>Coupon 2</td>
<td>398.3</td>
<td>29.0</td>
</tr>
<tr>
<td>Coupon 3</td>
<td>336.6</td>
<td>25.5</td>
</tr>
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<td><strong>Average</strong></td>
<td><strong>364.9</strong></td>
<td><strong>27.3</strong></td>
</tr>
<tr>
<td><strong>Standard Deviation</strong></td>
<td><strong>31.2</strong></td>
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<th>Sheet Piles for specimens 6 to 9</th>
<th>Tensile Strength (MPa)</th>
<th>Elastic Modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon 1</td>
<td>574.4</td>
<td>24.4</td>
</tr>
<tr>
<td>Coupon 2</td>
<td>561.1</td>
<td>26.8</td>
</tr>
<tr>
<td>Coupon 3</td>
<td>464.9</td>
<td>26.4</td>
</tr>
<tr>
<td><strong>Average</strong></td>
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</tr>
<tr>
<td><strong>Standard Deviation</strong></td>
<td><strong>59.8</strong></td>
<td><strong>1.3</strong></td>
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</tbody>
</table>

### Comments on Precision of Test Setup

1. **Location of Supports and Loading Lines:** Support locations and loading lines were measured relative to the supports using a conventional retractable measuring tape and marked using a permanent marker with a line thickness of approximately 3mm. As such, the size of the constant moment zone and the shear spans were likely within approximately 3mm of their designed dimensions.

2. **Normalization of Data Acquisition:** Loading beams and the spreader beam were assumed to have zero mass, and hence the data acquisition system was zeroed after these items were put in place. This would likely have altered the load by approximately 0.5kN, and would have added minute strains and deflections to the system as well. Because the GFRP forms were treated as stay-in-place structural formwork, the self weight of the concrete was resisted entirely by the GFRP forms. As such, the forms had undergone some initial strains and deflections due
to self weight prior to the application of any super-imposed loads, whereas the concrete had zero strain. This strain discontinuity would have a minor effect on the accuracy of curvature calculations. Strain gauges were applied to the GFRP after these strains occurred, and thus these initial strains were not recorded or accounted for in any way. Due to the minute magnitude of these initial strains relative to the strains that the sections later endured as superimposed loads were applied, it was decided that the effect of these initial strains was negligible.
APPENDIX 2(a):

FORTRAN90 Code for Analytical Model of Slab Specimens in Positive Bending

PROGRAM BONDSLAB6

IMPLICIT NONE
REAL :: TSTRAIN, CSTRAIN !Tensile and compressive strains at extreme fibres
REAL :: STRAINY !Strain at a given height Y
REAL :: YBAR, D1, D2 !Height of neutral axis, and two pointers used to find YBAR
REAL :: H, B, TPLATE !Total depth of section, width of section, and thickness of FRP plate
REAL :: HBOND !Total width of bonded area between concrete and FRP
REAL :: XCRACK !Distance from each support to region where cracking moment is surpassed
REAL :: FRPFORCECRACK !Total force in the FRP sheet at the location of the crack
REAL :: NEWYBAR !Height of neutral axis prior to cracking
REAL :: M, MR, CURV !Moment, maximum moment of resistance, and curvature
REAL :: ERROR !Allowable error in the location of the neutral axis
REAL :: SUMFORCE !Sum of all forces in the section
REAL :: THICK !Thickness of each layer
REAL :: RHO !Reinforcement ratio
REAL :: FCPRIME, FC !Specified compression and tensile strength of concrete
REAL :: EC, ECPRIME !Initial modulus of concrete and modulus at FCPRIME
REAL :: ECN !Effective modulus of concrete for use in modulus ratio
REAL :: NE !Modulus ratio
REAL :: STRAINFCPRIME !Strain in concrete at FCPRIME
REAL :: CDENS !Density of concrete
REAL :: FSCT, FSCC !Tension and compression failure strains for concrete
REAL :: N, K !Coefficients for determining concrete properties
REAL :: ALPHA1, ALPHA2 !Coefficients for tension stiffening
REAL :: FFPRIME, FF !Specified compression and tensile strength of FRP
REAL :: EFPRIME, EF !Specified compression and tensile modulus of FRP
REAL :: FST, FSFC !Tension and compression failure strains for FRP
REAL :: FRPFORCE !Total longitudinal force in FRP
REAL :: FBOND, BONDSTRENGTH !Bond stress and strength between FRP and concrete
REAL :: NEWFBOND !Bond stress in uncracked regions
REAL :: VC !Shear strength of concrete
REAL :: BONDFAILURE1, BONDFAILURE2, MLOAD, SHEARFAILURE, FAILURE !Loads at which bond failure, flexure failure, and shear failure occur
REAL :: A, SPAN !Shear span and clear span of the beam
REAL :: ENDLENGTH !Length of the overhangs beyond the support at each end of the beam
REAL :: LOAD, V !Total applied load on the beam, and resultant vertical shear in each shear span
REAL :: IXXT !Transformed second moment of inertia of section
REAL :: NEWIXXT !Transformed second moment of inertia of new uncracked section
REAL :: QFRP !First moment of area of FRP about neutral axis
REAL :: NEWQFRP !First moment of area of FRP about neutral axis of new uncracked section
REAL, PARAMETER :: PI = 3.141592654
REAL, PARAMETER :: NUMLAYER = 1000 !Total number of layers used in analysis
INTEGER :: I,Q,L,CR   REAL :: STRESSC, STRESSF !Stress in concrete and FRP layers respectively
OPEN(1,FILE='BONDSLAB6.TXT') !File in which moment curvature plot is output

H = 161.4  B = 400.0  BBOND = 380.0  A = 975  SPAN = 2200  ENDLENGTH = 110
XCRACK = SPAN/2  MCR = 0
CR = 1  IXXT = 0  QFRP = 0  NEWIXXT = 0  NEWQFRP = 0
FRPFORCE = 0
BONDFAILURE1 = 10**10  BONDFAILURE2 = 10**10  MLOAD = 0  SHEARFAILURE = 10**10
TPLATE = 9.525  FFPRIME = 137.5  FF = 137.5  EFPRIME = 18000.0  EF = 18000.0
RHO = TPLATE/H  CDENS = 2400  FCPRIIME = 37  VC = 0.2 * (FCPRIME**0.5) * B * (H-TPLATE)
FC = 0.62 * (FCPRIME**0.5)  EC = 4750*(FCPRIME**0.5)  FSCC = 0.003
FSCT = FC / EC  N = 0.8 + (FCPRIME / 17)  K = 0.67 + (FCPRIME / 62)
STRAINPRIME = (FCPRIME / EC) * (N / (N - 1))
IF (K < 1) THEN  K = 1  ENDF
ALPHA1 = 1  ALPHA2 = 1
FSFT = FF / EF  FSFC = FFPRIME / EFPRIME  BONDSTRENGTH = 1
ERROR = H / 50000
THICK = H / NUM LAYER
C STRAIN = 0
T STRAIN = 0
MR = 0
WRITE (1,100) "MOMENT", "CURVATURE", "MAX TENSILE STRAIN", "MAX COMPRESSION STRAIN", "YBAR", "IXX", "BOND STRESS" !Places headings on the document
100 format (A6,,A9,,A18,,A22,,A4,,A3,,A11)
DO WHILE (T STRAIN < FSFT .and. ABS(C STRAIN) < 2*FSCC) !Iterates to find moment vs curvature relationship (tries many tensile strains)
   D1 = 0
   D2 = H
   YBAR = (D1 + D2) / 2
   DO WHILE (D2 - D1 > ERROR) !Iterates to determine neutral axis location for a given tensile strain
      T STRAIN = YBAR * C STRAIN / (YBAR - H)
      FRPFORCE = 0
      SUMFORCE = 0
      M = 0
      IXXT = 0
      QFRP = 0
      DO I = 0, NUM LAYER - 1 !Sums forces and moments from each layer in the section
         STRAINY = T STRAIN * (1 - ((I*THICK) + (THICK/2))/YBAR)) !Strain at height of layer
         IF ((I*THICK) + (THICK/2) >= YBAR) THEN !If we are looking above the neutral axis (compression)
            STRESSF = EFPRIME * STRAINY !Stress in FRP
            STRESSC = 0 - (FCPRIME * (N * (ABS(STRAINY)/STRAINCP)**(N*K))/ (N-1+(ABS(STRAINY)/STRAINCP)**(N*K))) !Stress in concrete
            IF (ABS(STRAINY) >= FSCC) THEN
               STRESSC = 0
               ENDIF
            IF (ABS(STRAINY) >= FSFC) THEN
               STRESSF = 0
               ENDIF
            ELSEIF ((I*THICK) + (THICK/2) < YBAR) THEN !If we are looking below the neutral axis (tension)
               STRESSF = EF * STRAINY !Stress in FRP
               STRESSC = STRAINY * EC !Stress in concrete
               IF (ABS(STRAINY) >= FSCT) THEN
                  STRESSC = (ALPHA1*ALPHA2*FC) / (1 + (500*(STRAINY-FSCT)**0.5)) !Tension stiffening stress curve for concrete
                  ENDIF
            ELSEIF ((I*THICK) + (THICK/2) >= TPLATE) THEN !If we are looking at a layer within the concrete
               STRESSF = 0
            ELSEIF ((I*THICK) + (THICK/2) < TPLATE) THEN !If we are looking at a layer within the FRP
               STRESSC = 0
               NE = EF/EC
               IF (STRESSF == 0) THEN
                  NE = 0
                  END IF
               QFRP = QFRP + (THICK*B*NE*(ABS((I*THICK)-YBAR)))
               FRPFORCE = FRPFORCE + (B * THICK)*STRESSF
            ENDIF
      ENDIF
   ENDWHILE
   YBAR = (D1 + D2) / 2
   DO WHILE (D2 - D1 > ERROR) !Iterates to find moment vs curvature relationship (tries many tensile strains)
      T STRAIN = YBAR * C STRAIN / (YBAR - H)
      FRPFORCE = 0
      SUMFORCE = 0
      M = 0
      IXXT = 0
      QFRP = 0
      DO I = 0, NUM LAYER - 1 !Sums forces and moments from each layer in the section
         STRAINY = T STRAIN * (1 - ((I*THICK) + (THICK/2))/YBAR)) !Strain at height of layer
         IF ((I*THICK) + (THICK/2) >= YBAR) THEN !If we are looking above the neutral axis (compression)
            STRESSF = EFPRIME * STRAINY !Stress in FRP
            STRESSC = 0 - (FCPRIME * (N * (ABS(STRAINY)/STRAINCP)**(N*K))/ (N-1+(ABS(STRAINY)/STRAINCP)**(N*K))) !Stress in concrete
            IF (ABS(STRAINY) >= FSCC) THEN
               STRESSC = 0
               ENDIF
            IF (ABS(STRAINY) >= FSFC) THEN
               STRESSF = 0
               ENDIF
            ELSEIF ((I*THICK) + (THICK/2) < YBAR) THEN !If we are looking below the neutral axis (tension)
               STRESSF = EF * STRAINY !Stress in FRP
               STRESSC = STRAINY * EC !Stress in concrete
               IF (ABS(STRAINY) >= FSCT) THEN
                  STRESSC = (ALPHA1*ALPHA2*FC) / (1 + (500*(STRAINY-FSCT)**0.5)) !Tension stiffening stress curve for concrete
                  ENDIF
            ELSEIF ((I*THICK) + (THICK/2) >= TPLATE) THEN !If we are looking at a layer within the concrete
               STRESSF = 0
            ELSEIF ((I*THICK) + (THICK/2) < TPLATE) THEN !If we are looking at a layer within the FRP
               STRESSC = 0
               NE = EF/EC
               IF (STRESSF == 0) THEN
                  NE = 0
                  END IF
               QFRP = QFRP + (THICK*B*NE*(ABS((I*THICK)-YBAR)))
               FRPFORCE = FRPFORCE + (B * THICK)*STRESSF
            ENDIF
      ENDIF
   ENDWHILE
   WRITE (1,100) "MOMENT", "CURVATURE", "MAX TENSILE STRAIN", "MAX COMPRESSION STRAIN", "YBAR", "IXX", "BOND STRESS" !Places headings on the document
100 format (A6,,A9,,A18,,A22,,A4,,A3,,A11)
ECN = ABS(STRESSC / STRAINY)
NE = ECN/EC
IF (STRESSC == 0) THEN
NE = 0
END IF
IF (STRAINY <= FSCT) THEN
NE = 0
END IF
IXXT = IXXT + ABS(THICK*B*NE*(((I*THICK)-YBAR)**2))
SUMFORCE = SUMFORCE + (B * THICK)*STRESSC + STRESSF
M = M - ((I*THICK) + (THICK/2))*(B * THICK)*STRESSC + STRESSF
ENDDO
CURV = TSTRAIN / YBAR
IF (SUMFORCE < 0) THEN
D1 = YBAR
YBAR = (YBAR + D2) / 2
ELSEIF (SUMFORCE > 0) THEN
D2 = YBAR
YBAR = (YBAR + D1) / 2
ELSE
EXIT
ENDIF
ENDDO
IF (NEWQFRP == 0) THEN
NEWQFRP = QFRP
ENDIF
IF (NEWIXXT == 0) THEN
NEWIXXT = IXXT
ENDIF
IF (IXXT <= 0.8*NEWIXXT AND MCR == 0) THEN
MCR = M
NEWYBAR = YBAR
ENDIF
IF (IXXT <= 0.8*NEWIXXT) THEN
XCRACK = A * MCR/M
CR = CR + 1 !Counts iterations beyond the cracking moment
ENDIF
IF (M > MR) THEN
MR = M
ENDIF
MLOAD = 2*MR/A
LOAD = 2*M/A
V = LOAD/2
FRPFORCECRACK = TPLATE*B*MCR*(NEWYBAR-(TPLATE/2))/NEWIXXT
IF ((FRPFORCECRACK-FRPFORCE)/2 >= (BONDSTRENGTH * BBOND * (XCRACK + ENDLENGTH)) AND BONDFAILURE2 == 10**10) THEN !!!!!!!!!!!!!!!!
BONDFAILURE2 = LOAD
ENDIF
WRITE (1,150) M, CURV, TSTRAIN, CSTRAIN, YBAR, IXXT
CSTRAIN = CSTRAIN - 0.00001

ENDDO
CSTRAIN = 0
TSTRAIN = 0
NEWXXY = 0
NEWQFRP = 0
CR = 1
MCR = 0

DO WHILE (TSTRAIN < FSFT .and. ABS(CSTRAIN) < 2*FSCC) !Iterates to find moment vs curvature relationship (tries many tensile strains)
  D1 = 0
  D2 = H
  YBAR = (D1 + D2) / 2
  DO WHILE (D2 - D1 > ERROR)
    TSTRAIN = YBAR * CSTRAIN / (YBAR - H)
    FRPFORCE = 0
    SUMFORCE = 0
    M = 0
    IXXT = 0
    QFRP = 0
    DO I = 0, NUMLAYER - 1 !Sums forces and moments from each layer in the section
      STRAINY = TSTRAIN * (1 - (((I*THICK) + (THICK/2))/YBAR)) !Strain at height of layer
      IF ((I*THICK) + (THICK/2) >= YBAR) THEN !If we are looking above the neutral axis (compression)
        STRESSF = EFPRIME * STRAINY !Stress in FRP
        STRESSC = 0 - (FCPRIME * (N * (ABS(STRAINY)/STRAINCPRIME) / (N-1+(ABS(STRAINY)/STRAINCPRIME)**(N*K)))) !Stress in concrete
        IF (ABS(STRAINY) >= FSCC) THEN
          STRESSC = 0
        ENDIF
        IF (ABS(STRAINY) >= FSFC) THEN
          STRESSF = 0
        ENDIF
      ELSEIF ((I*THICK) + (THICK/2) < YBAR) THEN !If we are looking below the neutral axis (tension)
        STRESSF = EF * STRAINY !Stress in FRP
        STRESSC = STRAINY * EC !Stress in concrete
        IF (ABS(STRAINY) >= FSCT) THEN
          STRESSC = 0
        ENDIF
      ELSEIF (((I*THICK) + (THICK/2) < TPLATE) THEN !If we are looking at a layer within the FRP
        NE = EF/EC
        IF (STRESSF == 0) THEN
          NE = 0
        END IF
        QFRP = QFRP + (THICK*B*NE*(ABS((I*THICK)-YBAR)))
        FRPFORCE = FRPFORCE + (B * THICK)*STRESSF
      ELSEIF (((I*THICK) + (THICK/2) >= TPLATE) THEN !If we are looking at a layer within the concrete
        STRESSF = 0
      ENDIF
    END DO
  END DO
END DO
ECN = ABS(STRESSC / STRAINY)
NE = ECN / EC
IF (STRESSC == 0) THEN
   NE = 0
END IF
IF (STRAINY >= FSCT) THEN
   NE = 0
END IF
ENDIF
IXXT = IXXT + ABS(THICK*B*NE*(((I*THICK)-YBAR)**2))
SUMFORCE = SUMFORCE + (B * THICK)*STRESSC + STRESSF
M = M - (I*THICK) - (THICK**2)*(B * THICK)*((STRESSC + STRESSF))
ENDDO
CURV = TSTRAIN / YBAR
IF (SUMFORCE < 0) THEN
   D1 = YBAR
   YBAR = (YBAR + D2) / 2
ELSEIF (SUMFORCE > 0) THEN
   D2 = YBAR
   YBAR = (YBAR + D1) / 2
ELSE
   EXIT
ENDIF
ENDDO
IF (NEWQFRP == 0) THEN
   NEWQFRP = QFRP
ENDIF
IF (NEWIXXT == 0) THEN
   NEWIXXT = IXXT
   NEWYBAR = YBAR
ENDIF
IF (IXXT <= 0.8*NEWIXXT and. MCR == 0) THEN
   MCR = M
   NEWYBAR = YBAR
ENDIF
IF (IXXT <= 0.8*NEWIXXT) THEN
   XCRACK = A * MCR/M
   CR = CR + 1 !Counts iterations beyond the cracking moment
ENDIF
LOAD = 2*M/A
V = LOAD/2
NEWFBOND = V**NEWQFRP*(NEWIXXT*BBOND) !VQh!!!!!!!!!!!!
FBOND = V**QFRP(IXXT**BBOND) !VQh!!!!!!!!!!!!
IF (FBOND >= BONDFRAG and. BONDFAILURE1 == 10**10) THEN
   BONDFAILURE1 = LOAD
ENDIF
FRPFORCECRACK = TPLATE*B*MCR*(NEWYBAR-(TPLATE/2))/NEWIXXT
IF ((FRPFORCE*CRACK+FRPFORCE)/2 >= (BONDSTRENGTH * BBOND * (XCRACK + ENDLENGTH)) and. BONDFAILURE2 == 10**10) THEN !!!!!!!!!!!!!!!!

BONDFAILURE2 = LOAD
ENDIF

SHEARFAILURE = VC * 2

WRITE (1,165) M, CURV, TSTRAIN, CSTRRAIN, YBAR, IXXT, FBOND, MCR

CSTRRAIN = CSTRRAIN - 0.00001

ENDDO

IF (MLOAD <= BONDFAILURE1 and. MLOAD <= SHEARFAILURE and. MLOAD <= BONDFAILURE2) THEN  
FAILURE = MLOAD
ELSEIF (MLOAD <= BONDFAILURE1 and. MLOAD <= BONDFAILURE2 and. MLOAD <= SHEARFAILURE) THEN  
FAILURE = MLOAD
ELSEIF (SHEARFAILURE <= MLOAD and. SHEARFAILURE <= BONDFAILURE1 and. SHEARFAILURE <= BONDFAILURE2) THEN  
FAILURE = BONDFAILURE
ELSEIF (SHEARFAILURE <= MLOAD and. SHEARFAILURE <= BONDFAILURE1 and. SHEARFAILURE <= BONDFAILURE2) THEN  
FAILURE = BONDFAILURE
ELSEIF (BONDFAILURE1 <= MLOAD and. BONDFAILURE1 <= BONDFAILURE2 and. BONDFAILURE1 <= SHEARFAILURE) THEN  
FAILURE = BONDFAILURE1
ELSEIF (BONDFAILURE2 <= MLOAD and. BONDFAILURE2 <= BONDFAILURE1 and. BONDFAILURE2 <= SHEARFAILURE) THEN  
FAILURE = BONDFAILURE2
ENDIF

MU = A*(FAILURE/2)

WRITE (1,175) "_____________________________________________
175 format (A45)

WRITE (1,200) “Failure Moment Mu”, MU
200 format (A17,,F21.4)

WRITE (1,250) “Failure Load”, FAILURE
250 format (A12,,F26.4)

WRITE (1,275) "_____________________________________________
275 format (A45)

WRITE (1,300) “Shear Failure Load”, SHEARFAILURE
300 format (A18,,F20.4)

WRITE (1,350) “Initial Bond Failure Load”, BONDFAILURE1
350 format (A25,,F23.4)

WRITE (1,400) “Ultimate Bond failure Load”, BONDFAILURE2
400 format (A26,,F22.4)

WRITE (1,450) “Flexure Failure Load”, MLOAD
450 format (A20,,F18.4)

WRITE (1,500) “Flexure Failure Moment”, MR
500 format (A22,,F16.4)

ENDPROGRAM BONDSLAB6
APPENDIX 2(b):

FORTRAN90 Code for Analytical Model of Type 2 Girder Specimens in Positive Bending (specimen 4)

PROGRAM SHEETPILEHYBRIDZONE1

IMPLICIT NONE

REAL :: TSTRAIN, CSTRAIN !Tensile and compressive strains at extreme fibres
REAL :: STRAINY !Strain at a given height Y
REAL :: YBAR, D1, D2 !Height of neutral axis, and two pointers used to find YBAR
REAL :: MCR !Cracking moment of section
REAL :: XCRACK !Distance from each support to region where cracking moment is surpassed
REAL :: FRPFORCECRACK !Total force in the FRP sheet at the location of the crack
REAL :: NEWYBAR !Height of neutral axis prior to cracking
REAL :: H, BF !Total depth of section, and Total width of section (one T-Beam)
REAL :: DLAB !Thickness (depth) of concrete slab
REAL :: HVOID !Width of void within concrete
REAL :: DPILE !Total depth of FRP sheetpile section
REAL :: TFLANGEPILE, TWEBPILE !Thickness of the FRP sheetpile section in the flanges and the webs
REAL :: HBW, BWT !Total outside width of the web of the T-beam section at the bottom and top of the web
REAL :: WELHANGLE !Angle of the webs of the FRP sheetpile section (degrees from vertical)
REAL :: PILEWIDTH !Total width of FRP sheetpile section (centre-to-centre spacing of corrugations)
REAL :: WFLINSIDE !Inside width of lower flange
REAL :: M, MR, CURV !Moment, maximum moment of resistance, and curvature
REAL :: ERROR !Allowable error in the location of the neutral axis
REAL :: SUMFORCE !Sum of all forces in the section
REAL :: THICK !Thickness of each layer
REAL :: WFRP !Width of FRP layer
REAL :: CONCRETE !Width of concrete layer
REAL :: FCPRIME, FC !Specified compression and tensile strength of concrete
REAL :: EC, ECPRIME !Initial modulus of concrete and modulus at FCPRIME
REAL :: ECN !Effective modulus of concrete for use in modulus ratio
REAL :: NEC !Modulus ratio for concrete
REAL :: NEF !Modulus ratio for FRP
REAL :: NFELS !Specific modulus ratio for FRP at lower flange of section
REAL :: STRAINCRIME !Strain in concrete at FCPRIME
REAL :: CDENS !Density of concrete
REAL :: FSCT, FSCC !Tension and compression failure strains for concrete
REAL :: N, K !Coefficients for determining concrete properties
REAL :: ALPHA1, ALPHA2 !Coefficients for tension stiffening
REAL :: RCZONETHICK !Thickness of concrete layer that experiences tension stiffening
REAL :: WPLZONE !Width of zone that does not experience tension stiffening
REAL :: FFPRIME, EF !Specified compression and tensile modulus of FRP sheetpile section
REAL :: FSFT, FSFC !Tension and compression failure strains for FRP sheetpile section
REAL :: FRPFORCE !Total longitudinal force in FRP
REAL :: UFBOND !Percent of upper flange that is covered with bond mechanism
REAL :: LFBOND  !Percent of lower flange that is covered with bond mechanism
REAL :: WBOND  !Percent of web that is covered with bond mechanism (must be either 0% or 100%)
REAL :: TOTBOND  !Total bond shear force between GFRP and concrete
REAL :: BONDSTRENGTH  !Bond strength between FRP and concrete
REAL :: FBONDDB, FBONDWEB, FBONDBOTTOM, FBONDBOTTOMMULT  !Bond stress below concrete slab, at upper-most bonded region of web, and at lower flange
REAL :: FBONDWEBAVG  !Average bond stress in the web
REAL :: NEWFBOND  !Bond stress in uncracked regions
REAL :: VC  !Shear strength of concrete
REAL :: BONDFAILURE1, BONDFAILURE2, BONDFAILURE3, MLOAD, SHEARFAILURE, FAILURE  !Loads at which bond failure, flexure failure, and shear failure occur
REAL :: A, SPAN  !Shear span and clear span of the beam
REAL :: ENDLENGTH  !Length of the overhangs beyond the support at each end of the beam
REAL :: LOAD, V  !Total applied load on the beam, and resultant vertical shear in each shear span
REAL :: IXXT  !Transformed second moment of inertia of section
REAL :: NEWIXXT  !Transformed second moment of inertia of new uncracked section
REAL :: QSLAB  !First moment of area of concrete slab about neutral axis
REAL :: QFRP  !First moment of area of sheet pile about neutral axis
REAL :: NEWQSLAB  !First moment of area of concrete slab about neutral axis of new uncracked section
REAL, PARAMETER :: PI = 3.141592654  REAL, PARAMETER :: NUMLAYER = 1000  !Total number of layers used in analysis
INTEGER :: NQL, CR
REAL :: STRESSC, STRESSF  !Stress in concrete and FRP layers respectively
OPEN(1,FILE='SHEETPILEHYBRIDZONE1.TXT')  !File in which moment curvature plot is output

PILEWIDTH = 609.6
DPILE = 254
TFIANGLEPILE = 8.99
TWEBPILE = 7.62
WEBANGLE = 20
WEBANGLE = 2*PI*WEBANGLE/360
BWB = 208.28
BWT = BWB + 2*(DPILE - TFLANGEPILE)*TAN(WEBANGLE)
WLFINSIDE = (BWB - 2*TWEBPILE/COS(WEBANGLE)) + 2*TFLANGEPILE*TAN(WEBANGLE)
A = 1350
SPAN = 3100
ENDLENGTH = 125
XCRACK = SPAN/2
MCR = 0
CR = 1
IXXT = 0
QSLAB = 0
QFRP = 0
QFLANGE = 0
NEWIXXT = 0
NEWQSLAB = 0
FRPFORCE = 0
BONDFAILURE1 = 10**10
BONDFAILURE2 = 10**10
BONDFAILURE3 = 10**10
MLOAD = 0
SHEARFAILURE = 10**10

FFPRIME = 345
FF = 517
EFPRIME = 26200.0
EF = 26200.0

FSFT = FF / EF
FSFC = FFPRIME / EFPRIME

dslab = 70
bvoid = 0

cdens = 2400
fcp = 38.71
fc = 0.63 * (fcprime * 0.5)
ec = (3300 * (fcprime * 0.5) + 6900) * (cdens/2300)**(1.5)
fscc = 0.0035
fsct = fc / ec

n = 0.8 * (fcprime / 17)
k = 0.67 * (fcprime / 62)
strainprime = (fcprime / ec) * (n / (n - 1))
if (k < 1) then
  k = 1
endif

alpha1 = 1
alpha2 = 1

rczonenethick = (7.5 * tflangepile/2) + (tflangepile/2) / collins and mitchell figure 5-6 (isbn 0-9691816-6-3)

h = dipile + dslab
bf = pilewidth

bondstrength = 1
error = h / 50000
thick = h / numlayer

ubond = 1
lfbond = 1 - (bvoid((bw - 2*twebblepile*cos(webangle)) + 2*tflangetube*tan(webangle)))
wbond = 1

totbond = 0

cstrain = 0
tstrain = 0

mr = 0

write (1,100) "moment", "curvature", "max tensile strain", "max compressive strain", "ybar" / places headings on the document

100 format (a6,",",a9, ",", a18, ",", a22, ",", a4)
do while (tstrain < fsft .and. abs(cstrain) < 2*fscc) !iterates to find moment vs curvature relationship (tries many tensile strains)
  d1 = 0
  d2 = h
  ybar = (d1 + d2) / 2
  do while (d2 - d1 > error) !iterates to determine neutral axis location for a given tensile strain
TSTRAIN = YBAR * CSTRAIN / (YBAR - H)

FRPFORCE = 0
SUMFORCE = 0
M = 0
IXXT = 0
QSLAB = 0
QFRP = 0
QLFLANGE = 0

DO I = 0, NUMLAYER - 1 ! Sums forces and moments from each layer in the section

STRAINY = TSTRAIN * (1 - (((I*THICK) + (THICK/2))/YBAR)) ! Strain at height of layer

IF ((I*THICK) + (THICK/2) >= YBAR) THEN ! If we are looking above the neutral axis (compression)
STRESSF = EFPRIME * STRAINY ! Stress in FRP
STRESSC = 0 - (FCPRIME * (N * (ABS(STRAINY)/STRAINCPRIME) / (N-1+(ABS(STRAINY)/STRAINCPRIME)**(N*K)))) ! Stress in concrete
IF (ABS(STRAINY) == FSCT) THEN
STRESSC = 0
ENDIF
ELSEIF ((I*THICK) + (THICK/2) < YBAR) THEN ! If we are looking below the neutral axis (tension)
STRESSF = EF * STRAINY ! Stress in FRP
STRESSC = STRAINY * EC ! Stress in concrete
IF (ABS(STRAINY) == FSCT) THEN
STRESSC = (ALPHA1*ALPHA2*FC) / (1 + (500*(STRAINY-FSCT))**0.5) ! Tension stiffening stress curve for concrete
ENDIF

NEF = EF/EC
IF (STRESSF == 0) THEN
NEF = 0
ENDIF
NEC = ABS(STRESSC / STRAINY)
IF (STRESSC == 0) THEN
NEC = 0
ENDIF

IF ((I*THICK) + (THICK/2) < TFLANGEPILE) THEN ! If we are looking at a layer within the lower flange of the FRP sheetpile section
WCONCRETE = 0
WFRP = BWB + 2*((I*THICK) + (THICK/2))*TAN(WEBANGLE)
QLFLANGE = QLFLANGE + ABS(THICK*WFRP*NEF*((I*THICK)-YBAR))
NEFLF = NEF
ELSEIF ((I*THICK) + (THICK/2) >= TFLANGEPILE .and. (I*THICK) + (THICK/2) < (DPILE - TFLANGEPILE)) THEN ! If we are looking at a layer within the web
WCONCRETE = (BWB - 2*TWEBPILE/COS(WEBANGLE)) + 2*((I*THICK) + (THICK/2))*TAN(WEBANGLE) - TFLANGEPILE
WPLZONE = WCONCRETE - (2*RCZONETHICK/COS(WEBANGLE))
ELSEIF WCONCRETE - (2*RCZONETHICK/COS(WEBANGLE)) > 0 THEN
WPLZONE = WCONCRETE - (2*RCZONETHICK/COS(WEBANGLE))
ELSE
WPLZONE = 0
ENDIF

ENDIF

IF ((I*THICK) + (THICK/2) > RCZONETHICK + TFLANGEPILE .and. STRAINY >= FSCT) THEN
WCONCRETE = WCONCRETE - WPLZONE
ENDIF
WFRP = 2*TWEBPILE/COS(WEBANGLE)
ELSEIF ((I*THICK) + (THICK/2) >= (DPILE - TFLANGEPILE) .and. (I*THICK) + (THICK/2) < DPILE) THEN !If we are looking at a layer within the upper flange of the FRP sheetpile section
    WCONCRETE = (BWB - 2*TWEBPILE/COS(WEBANGLE)) + 2*((I*THICK) + (THICK/2))*TAN(WEBANGLE)-BVOID
    WFRP = ((PILEWIDTH - BWT) + 2*TWEBPILE/COS(WEBANGLE)) - 2*((I*THICK) + (THICK/2) - (DPILE - TFLANGEPILE))*TAN(WEBANGLE)
ELSEIF ((I*THICK) + (THICK/2) >= DPILE) THEN !If we are looking at a layer within the concrete slab
    WCONCRETE = BF
    WFRP = 0
    QSLAB = QSLAB + ABS(THICK*WCONCRETE*NEC*((I*THICK)-YBAR))
ENDIF
QFRP = QFRP + ABS(THICK*WFRP*NEF*((I*THICK)-YBAR))
IXXT = IXXT + ABS(THICK*WCONCRETE*NEC*(((I*THICK)-YBAR)**2)) + ABS(THICK*WFRP*NEF*(((I*THICK)-YBAR)**2))
SUMFORCE = SUMFORCE + THICK*WCONCRETE*NEC*(((I*THICK)-YBAR)**2) + (WFRP*STRESSF)
M = M - ((I*THICK) + (THICK/2))*THICK*((WCONCRETE*STRESSC) + (WFRP*STRESSF))
ENDIF
ENDDO
CURV = TSTRAIN / YBAR
IF (SUMFORCE < 0) THEN
    D1 = YBAR - YBAR + D2 / 2
ELSEIF (SUMFORCE > 0) THEN
    D2 = YBAR
    YBAR = (YBAR + D1) / 2
ELSE
    EXIT
ENDIF
ENDDO
ENDDO
IF (M > MR) THEN
    MR = M
ENDIF
V = M/A
LOAD = V*2
WRITE (1,150)  M, CURV, TSTRAIN, CSTRAIN, YBAR, LOAD
CSTRAIN = CSTRAIN .- 0.00001
ENDDO
CSTRAIN = 0
TSTRAIN = 0
NEWIXXT = 0
NEWQSLAB = 0
CR = 1
MCR = 0
DO WHILE (TSTRAIN < FSFT .and. ABS(CSTRAIN) < 2*FSCC) !Iterates to find moment vs curvature relationship (tries many tensile strains)
    D1 = 0
    D2 = H
    YBAR = (D1 + D2) / 2
    DO WHILE (D2 - D1 > ERROR) !Iterates to determine neutral axis location for a given tensile strain
        TSTRAIN = YBAR * CSTRAIN / (YBAR - H)
        FRPFORCE = 0
    END
SUMFORCE = 0
M = 0
IXXT = 0
QSLAB = 0
QFRP = 0
QLFLANGE = 0

DO I = 0, NUMLAYER - 1 !Sums forces and moments from each layer in the section

STRAINY = TSTRAIN * (1 - (((I*THICK) + (THICK/2))/YBAR)) !Strain at height of layer

IF ((I*THICK) + (THICK/2) >= YBAR) THEN !If we are looking above the neutral axis (compression)
  STRESSF = EFPRIME * STRAINY !Stress in FRP
  STRESSC = 0 - (FCPRIME * (N * (ABS(STRAINY)/STRAINCPRIME) / (N-1+(ABS(STRAINY)/STRAINCPRIME)**(N*K)))) !Stress in concrete
  IF (ABS(STRAINY) >= FSCC) THEN
    STRESSC = 0
  ENDIF
  IF (ABS(STRAINY) >= FSFC) THEN
    STRESSF = 0
  ENDIF
ELSEIF ((I*THICK) + (THICK/2) < YBAR) THEN !If we are looking below the neutral axis (tension)
  STRESSF = EF * STRAINY !Stress in FRP
  STRESSC = STRAINY * EC !Stress in concrete
  IF (ABS(STRAINY) >= FSCT) THEN
    STRESSC = 0
  ENDIF
ENDIF

NEF = EF/EC
IF (STRESSF == 0) THEN
  NEF = 0
ENDIF
IF (STRAINY >= FSFT) THEN
  NEF = 0
ENDIF
ECN = ABS(STRESSC / STRAINY)
NEC = ECN/EC
IF (STRESSC == 0) THEN
  NEC = 0
ENDIF

IF ((I*THICK) + (THICK/2) < TFLANGEPILE) THEN !If we are looking at a layer within the lower flange of the FRP sheetpile section
  WCONCRETE = 0
  WFRP = BWB + 2*((I*THICK) + (THICK/2))*TAN(WEBANGLE)
  QLFLANGE = QLFLANGE + ABS(THICK*WFRP*NEF*((I*THICK)-YBAR))
  NEFLF = NEF
ELSEIF ((I*THICK) + (THICK/2) >= TFLANGEPILE .and. (I*THICK) + (THICK/2) < (DPILE - TFLANGEPILE)) THEN !If we are looking at a layer within the web
  WCONCRETE = (BWB - 2*TWEBPILE/COS(WEBANGLE)) + 2*((I*THICK) + (THICK/2))*TAN(WEBANGLE)-BVOID
  WFRP = 2*TWEBPILE/COS(WEBANGLE)
ELSEIF ((I*THICK) + (THICK/2) >= (DPILE - TFLANGEPILE) .and. (I*THICK) + (THICK/2) < DPILE) THEN !If we are looking at a layer within the upper flange of the FRP sheetpile section
  WCONCRETE = (BWB - 2*TWEBPILE/COS(WEBANGLE)) + 2*(I*THICK) + (THICK/2)*TAN(WEBANGLE)-BVOID
  WFRP = ((PILEWIDTH - BWT) + 2*TWEBPILE/COS(WEBANGLE)) - 2*(I*THICK) + (THICK/2) - (DPILE - TFLANGEPILE)*TAN(WEBANGLE)
ELSEIF ((I*THICK) + (THICK/2) >= DPILE) THEN !If we are looking at a layer within the concrete slab
  WCONCRETE = BF
  WFRP = 0
  QSLAB = QSLAB + ABS(THICK*WCONCRETE*NEC*((I*THICK)-YBAR))
ENDIF

QFRP = QFRP + ABS(THICK*WFRP*NEF*(I*THICK)-YBAR))
IXXT = IXXT + ABS(THICK*WCONCRETE*NEC*((I*THICK)-YBAR)**2) + ABS(THICK*WFRP*NEF*((I*THICK)-YBAR)**2)
SUMFORCE = SUMFORCE + THICK*(WCONCRETE*STRESSC + (WFRP*STRESSF))
FRPFORCE = FRPFORCE + THICK*WFRP*STRESSF
\[ M = M - ((I*THICK) + (THICK/2))*THICK*((WCONCRETE*STRESSC) + (WFRP*STRESSF)) \]

ENDDO

\[ \text{CURV} = \frac{\text{TSTRAIN}}{\text{YBAR}} \]

IF (SUMFORCE < 0) THEN

\[ D1 = \text{YBAR} \]
\[ \text{YBAR} = \frac{\text{YBAR} + D2}{2} \]
ELSEIF (SUMFORCE > 0) THEN

\[ D2 = \text{YBAR} \]
\[ \text{YBAR} = \frac{\text{YBAR} + D1}{2} \]
ELSE

EXIT
ENDIF

ENDIF

\[ \text{ENDDO} \]

IF (M > MR) THEN

MR = M
ENDIF

\[ V = \frac{M}{A} \]
\[ \text{LOAD} = V^2 \]

\[ \text{FBONDSLAB} = V^*\text{QSLAB(IXXT*PIECEWIDTH)} \]
\[ \text{FBONDWEBAVG} = (V^*\text{QFRP}(\text{IXXT} - (\text{WLFINSIDE} - \text{BVOID})^*(\text{QFLANGE}(\text{IXXT}*(2^*\text{TWEBPILE/COS(WEBANGLE)}) + (\text{WLFINSIDE} - \text{BVOID}))^2) - (\text{WLFINSIDE} - \text{BVOID}) + 2^*(\text{DPILE} - \text{TFLANGEPILE}/\text{COS(WEBANGLE)})) \]
\[ \text{FBONDBOTTOM} = V^*\text{QFLANGE}(\text{IXXT}*(2^*\text{TWEBPILE/COS(WEBANGLE)}) + (\text{WLFINSIDE} - \text{BVOID})) \]
\[ \text{FBONDWEBMULT} = V^*\text{QFRP}(\text{IXXT}*(\text{WLFINSIDE} - \text{BVOID})) \]

IF (FBONDSLAB >= BONDSTRENGTH \text{ and } BONDFAILURE1 == 10**10) THEN

BONDFAILURE1 = LOAD
ENDIF

ENDIF

IF (FBONDBOTTOM >= BONDSTRENGTH \text{ and } BONDFAILURE3 == 10**10) THEN

BONDFAILURE3 = LOAD
ENDIF

WRITE (1,175) M, CURV, TSTRAIN, CSTRAIN, YBAR, MR, LOAD, FBONDSLAB, FBONDBOTTOM, FBONDWEBMULT
CSTRAIN = CSTRAIN - 0.00001

ENDIF

WRITE (1,200) "Mr", MR
200 format (A2, F25.13)

WRITE (1,250) "Bond Failure Load 1", BONDFAILURE1
250 format (A19, F25.13)

ENDPROGRAM SHEETPILEHYBRIDZONE1