Shear Strengthening of Prestressed Hollow Core Slabs using Externally Bonded Glass Fibre Reinforced Polymer Sheets

Amr Alaa Abdelaal

University of Windsor

Follow this and additional works at: https://scholar.uwindsor.ca/etd

Recommended Citation
https://scholar.uwindsor.ca/etd/7340

This online database contains the full-text of PhD dissertations and Masters' theses of University of Windsor students from 1954 forward. These documents are made available for personal study and research purposes only, in accordance with the Canadian Copyright Act and the Creative Commons license—CC BY-NC-ND (Attribution, Non-Commercial, No Derivative Works). Under this license, works must always be attributed to the copyright holder (original author), cannot be used for any commercial purposes, and may not be altered. Any other use would require the permission of the copyright holder. Students may inquire about withdrawing their dissertation and/or thesis from this database. For additional inquiries, please contact the repository administrator via email (scholarship@uwindsor.ca) or by telephone at 519-253-3000 ext. 3208.
Shear Strengthening of Prestressed Hollow Core Slabs using Externally Bonded Glass Fibre Reinforced Polymer Sheets

By

Amr Alaa Abdelaal

A Thesis Submitted to the Faculty of Graduate Studies through the Department of Civil & Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Master of Applied Science at the University of Windsor

Windsor, Ontario, Canada

© 2017 Amr Abdelaal
Shear Strengthening of Prestressed Hollow Core Slabs using Externally Bonded Glass Fibre Reinforced Polymer Sheets

By

Amr Alaa Abdelaal

APPROVED BY:

____________________________________________

A. Edrisy
Department of Mechanical, Automotive and Materials Engineering

____________________________________________

F. Ghrib
Department of Civil & Environmental Engineering

____________________________________________

A. El Ragaby, Advisor
Department of Civil & Environmental Engineering

____________________________________________

S. Cheng, Co-Advisor
Department of Civil & Environmental Engineering

December 7, 2017
DECLARATION OF ORIGINALITY

I hereby certify that I am the sole author of this thesis and that no part of this thesis has been published or submitted for publication.

I certify that, to the best of my knowledge, my thesis does not infringe upon anyone’s copyright nor violate any proprietary rights and that any ideas, techniques, quotations, or any other material from the work of other people included in my thesis, published or otherwise, are fully acknowledged in accordance with the standard referencing practices. Furthermore, to the extent that I have included copyrighted material that surpasses the bounds of fair dealing within the meaning of the Canada Copyright Act, I certify that I have obtained a written permission from the copyright owner(s) to include such material(s) in my thesis and have included copies of such copyright clearances to my appendix.

I declare that this is a true copy of my thesis, including any final revisions, as approved by my thesis committee and the Graduate Studies office, and that this thesis has not been submitted for a higher degree to any other University or Institution.
ABSTRACT

Precast prestressed hollow core (PHC) slabs are widely used as floor deck systems. However, as extrusion is the most widely adopted manufacturing technique of PHC slabs, it is difficult to arrange shear reinforcement during casting using an extrusion machine. Therefore, the shear capacity of a PHC slab relies on the shear strength of plain concrete, which makes them prone to shear failure near the support. Traditional solutions to increase the shear capacity of PHC slabs would add more weight and cost. Externally bonded fibre reinforced polymers (FRP) sheets have been successfully used over the past 20 years as an efficient technique for strengthening both the flexural and shear capacities of reinforced concrete members. During the first two phases of this research project, externally bonded carbon fibre reinforced polymers (CFRP) sheets along the internal perimeter of the PHC slab voids have proven to considerably increase the shear capacity of PHC slabs by up to 40%. The objective of the current research, which is Phase III, is to investigate and compare the effectiveness of glass fibres reinforced polymers (GFRP) sheets to CFRP sheets used in Phase I and Phase II to enhance the shear capacity of PHC slabs. A total of 11 full size PHC slabs, 1219 mm wide, 4500 mm long and 305 mm thick, were tested in Phase III. The effects of fibre type, the prestressing level, the width of the GFRP sheets, and the installation procedures for the shear capacity of PHC slabs were investigated. The ultimate strength, deflection, strands’ slippage, concrete strain and FRP strain were recorded, analyzed and compared to the results obtained in the previous study. The testing results show significant enhancement in the shear capacity and the behaviour of the PHC slabs strengthened by externally bonded GFRP sheets along the internal perimeter of the PHC slab voids comparable to the effect of the CFRP sheets.

Keywords: GFRP; Prestressed Hollow Core Slabs: Shear Strengthening
DEDICATION

To my mother, father, and brother:

Amal Mowina, Alaa Abdelaal, and Ahmed Abdelaal

Thank you for everything.
ACKNOWLEDGEMENTS

I am very thankful to Dr. Amr El Ragaby, my Advisor, for his continuous help and support during my experimental work, analysis, and writing my thesis. I was also lucky to have Dr. Shaohong Cheng as my Co-Advisor. All her support during the program and valuable suggestions to my experimental work and thesis are appreciated.

I would like to thank Dr. Afsaneh Edrisy, the Outside Department Reader, for dedicating her time to read and review my thesis, and offer advice and recommendations.

I would like to express my appreciation to Dr. Faouzi Ghrib for helping me during the program and for taking the time to analyze my thesis and give me constructive feedback.

I would like to recognize the input made by Prestressed Concrete Inc. (PSI) in this project for supplying the experimental specimens.

I would also like to acknowledge the help I received during my laboratory work from all the technical staff.

Additionally, I would especially like to acknowledge the support I received from Mr. Matthew St. Louis.

Special thanks to my brother, Ahmed Abdelaal, for being available when needed during the experimental work and to all my friends who supported me when it was needed.

I am tremendously grateful to have my parents. I would not have reached this point in my life without their unlimited love, inspiration and support.
TABLE OF CONTENT

DECLARATION OF ORIGINALITY .......................................................................................... III

DEDICATION ........................................................................................................................... IV

ABSTRACT ............................................................................................................................ IV

ACKNOWLEDGEMENTS ........................................................................................................ VI

LIST OF TABLES .................................................................................................................. XII

LIST OF FIGURES ............................................................................................................... XIII

CHAPTER 1: INTRODUCTION ............................................................................................. 1

1.1 Background ..................................................................................................................... 1

1.2 Motivation ....................................................................................................................... 2

1.3 Objectives ....................................................................................................................... 5

CHAPTER 2: LITERATURE REVIEW ..................................................................................... 6

2.1 Web Shear Strength of PHC Slabs .................................................................................. 6

2.1.1 Limit state design (LSD) .......................................................................................... 7

2.1.1.1 Ultimate limit state (ULS) .................................................................................. 7

2.1.1.2 Serviceability limit state (SLS) ......................................................................... 7

2.2.1 Shear tension and compression failure in web ......................................................... 8

2.2.2 Factors affecting web shear capacity ........................................................................ 9

2.2.2.1 Depth of the member ......................................................................................... 9

2.2.2.2 Geometry of the cross section .......................................................................... 9
2.2.2.3 Axial stress due to prestressing forces ................................................................. 10
2.2.2.4 Tensile strength of concrete ................................................................................ 10
2.2.2.5 Shear lag of prestressing force .......................................................................... 11
2.2.2.6 Shear span to depth ratio (a/d) ........................................................................... 11

2.2 Previous Work Using FRP Sheets to Externally Strengthen Concrete Beams and PHC... 12
   Slabs ................................................................................................................................ 12
   2.2.1 Shear strengthening of RC beams using CFRP sheets ......................................... 12
   2.2.2 Shear strengthening of RC beams using GFRP sheets ......................................... 15
   2.2.3 Summary ............................................................................................................... 16
   2.2.4 Shear strengthening of PHC slabs using externally bonded FRP sheets ............... 17

2.3 Code Provisions and Guidelines for Shear Capacity of PHC Slabs ......................... 19
   2.3.1 ACI-318-14 code .................................................................................................. 19
   2.3.2 CSA-A23.3-14 code ............................................................................................ 20

2.4 Code Provisions and Guidelines for RC Members Strengthened in Shear using FRP .... 21
   2.4.1 ACI 440.2R-15 code .......................................................................................... 21
   2.4.2 CSA-S806-12 code ............................................................................................ 21

CHAPTER 3: EXPERIMENTAL PROGRAM ........................................................................ 22

3.1 Test Specimens .......................................................................................................... 22
3.2 Preparation of Test Specimens ................................................................................ 25
   3.2.1 Wet installation method ...................................................................................... 25
3.2.2 Precured installation method ........................................................................................ 25

3.3 Material Properties .............................................................................................................. 26

3.3.1 Concrete ........................................................................................................................ 26

3.3.2 Epoxy ............................................................................................................................ 26

3.3.3 GFRP sheets ................................................................................................................. 27

3.3.4 Prestressing strands ...................................................................................................... 27

3.4 Test Setup and Instrumentations ......................................................................................... 27

CHAPTER 4: TEST RESULTS AND ANALYSIS ..................................................................... 31

4.1 Effect of the Strengthening GFRP Sheets Width ................................................................. 31

4.1.1 Test observations .......................................................................................................... 32

4.1.2 Ultimate load ................................................................................................................ 34

4.1.3 Load deflection behaviour ............................................................................................ 35

4.1.4 Strain behaviour ............................................................................................................ 35

4.2 Effect of Prestressing Level ................................................................................................. 39

4.2.1 Test observations .......................................................................................................... 40

4.2.2 Ultimate load ................................................................................................................ 43

4.2.3 Load deflection behaviour ............................................................................................ 44

4.3 Effect of Installation Method of the GFRP Sheets .............................................................. 47

4.3.1 Test observations .......................................................................................................... 48

4.3.2 Ultimate load ................................................................................................................ 51

IX
REFERENCE................................................................................................................................. 84

VITA AUCTORIS........................................................................................................................................ 88
LIST OF TABLES

Table 3.1 Testing matrix ............................................................................................................... 24
Table 4.1 S2-Series specimens’ parameters.................................................................................. 32
Table 4.2 Test results of S2-Series slab specimens based on different strengthened widths ...... 34
Table 4.3 S1-Series, S2-Series and S3-Series specimens’ parameters ........................................ 39
Table 4.4 Ultimate loads of three strengthened specimens using 180° arc width GFRP sheets... 43
Table 4.5 Ultimate loads of two strengthened specimens using 150° arc width GFRP sheets .... 43
Table 4.6 S3-Series specimens’ parameters.................................................................................. 48
Table 4.7 Test results of S3-Series slab specimens ...................................................................... 52
Table 4.8 S2-Series specimens’ parameters.................................................................................. 57
Table 4.9 Test results of S2-Series slab specimens ................................................................. 58
Table 4.10 General items required for the strengthening process ............................................. 61
Table 4.11 Wet installation method quantities of GFRP sheets take off per specimen.............. 62
Table 4.12 Wet installation method quantities of CFRP sheets take off per specimen.............. 62
Table 4.13 Precured installation method quantities of GFRP sheets take out per specimen...... 63
Table 4.14 Precured installation method quantities of CFRP sheets take out per specimen...... 63
Table 5.1 ACI 318-15 and ACI 440-2r-15 shear prediction results and the experimental results for all specimens............................................................................................................ 66
Table 5.2 CSA-A23.3-14 and CSA-S806-12 shear prediction and the experimental results for all specimens ........................................................................................................................................ 68
LIST OF FIGURES

Figure 1.1 PHC slabs floor plan with filling................................................................. 2
Figure 2.1 PHC slab possible failure mechanisms under ULS (Pajari, 1991).................. 7
Figure 2.2 PHC slab possible failure mechanisms under SLS (Pajari, 1991)..................... 8
Figure 2.3 The shear critical point in the circular voids of a PHC slab (Yang, 1994)......... 10
Figure 2.4 The ratio of the shear span (a) to the depth (b)............................................. 12
Figure 2.5 Beam cracks after failure (Li et al., 2001).................................................... 13
Figure 2.6 CFRP sheets bonding layouts for all beams (Adhikary and Mutsuyshi, 2004).... 14
Figure 2.7 Shear strengthening techniques for single web hollow core slabs (Wu, 2015).... 18
Figure 2.8 PHC slab strengthened in shear by CFRP sheets (Meng, 2016)..................... 19
Figure 3.1 Cross section of slabs with different prestressing levels............................... 23
Figure 3.2 Schematics of GFRP installation position (units: mm)..................................... 23
Figure 3.3 Cross section of different GFRP sheets arc lengths ....................................... 24
Figure 3.4 PHC slab inner webs strengthened in shear by GFRP sheets.......................... 26
Figure 3.5 Overall experimental setup at loaded end..................................................... 28
Figure 3.6 Side view of the test setup (units: mm)......................................................... 29
Figure 3.7 The experiment full test setup........................................................................ 29
Figure 3.8 Cross section view of the test setup (units: mm)........................................... 30
Figure 4.1 S2-Series crack profile based on different strengthening width (units: mm).... 33
Figure 4.2 Concrete traces on two 150° arc width GFRP sheets after manually peeling them off the PHC slab web................................................................. 33
Figure 4.3 Concrete traces on two 120° arc width GFRP sheets after manually peeling them off the PHC slab web................................................................. 34

XIII
Figure 4.4 Load versus deflection near loading point based on different GFRP strengthening widths in S2-Series specimens ................................................................. 36

Figure 4.5 Load end slip behaviour based on different GFRP strengthening widths in S2-Series specimens ........................................................................................................ 37

Figure 4.6 Load versus longitudinal tensile strain in FRP sheets for S2-Series .................... 38

Figure 4.7 Crack profile of S1-Series, S2-Series and S3-Series specimens strengthened with 180° arc width GFRP sheets (units: mm) ................................................................. 41

Figure 4.8 Crack profile of S2-Series and S3-Series specimens strengthened with 150° arc width GFRP sheets (units: mm) .................................................................................. 41

Figure 4.9 Cracking patterns of S1-Series, S2-Series and S3-Series around the prestressing strands at loading end .................................................................................................... 42

Figure 4.10 Effect of prestressing level on the load deflection relation of S1-Series, S2-Series and S3-Series specimens strengthened with 180° arc width GFRP sheets .............. 45

Figure 4.11 Effect of prestressing level on the load deflection relation of S2-Series and S3-Series specimens strengthened with 150° arc width GFRP sheets ........................................... 45

Figure 4.12 Effect of prestressing level on the load strands slippage relation of S1-Series, S2-Series and S3-Series .............................................................................................. 46

Figure 4.13 Effect of prestressing level on the load strands slippage relation of S2-Series and S3-Series specimens strengthened with 150° arc width GFRP sheets .................... 47

Figure 4.14 Crack profile of S3-Series specimens based on the installation method (units: mm) 49

Figure 4.15 Comparison between the wet installation method and the precured installation method bonding after manually peeling off the sheets ........................................... 51

XIV
Figure 4.16 Effect of installation method on S3-Series specimen strengthened with 150° arc width GFRP sheets .............................................................. 53
Figure 4.17 Effect of installation method on S3-Series specimen strengthened with 120° arc width GFRP sheets .............................................................. 54
Figure 4.18 Effect of installation method on S3-Series specimen load end slip behaviour .......... 55
Figure 4.19 Effect of installation method on S3-Series specimen concrete compressive strain behaviour .................................................................................. 56
Figure 4.20 Crack profile of S2-Series (units: mm) ............................................................... 57
Figure 4.21 Load versus deflection near loading point for S2-Series specimens .................. 59
Figure 4.22 Load end slip behaviour for S2-Series specimens ............................................ 60
Figure 5.1 Ratio of the theoretical to experimental results based on ACI code for PHC slabs strengthened with GFRP sheets ........................................ 67
Figure 5.2 Ratio of the theoretical to experimental results based on CSA code for PHC slabs strengthened with GFRP sheets ........................................ 69
CHAPTER 1: INTRODUCTION

1.1 Background

Prestressed Hollow Core (PHC) slabs were first introduced as a structural element in early 1950s (Wijesundara et al., 2012). PHC slab design reduces the weight and the cost due to continuous voids along the span length. The prestressing strands combined with high depth/weight ratio increases the flexural capacity of PHC slabs, allowing for a longer span and higher live loads. Moreover, due to the hollow cores, PHC slabs have excellent fire resistance and heat transfer performance. Likewise, the cores inside the slabs are a practical place to install electricity and plumbing work without the need for adding a ceiling. Furthermore, the PHC slab system is an environmentally friendly construction element as it reduces the waste and noise of cast in place concrete since it is being casted in a manufactory. PHC slabs are well known for their ease and fast erection on site, which saves both time and cost. As precast elements, PHC slabs are available in different sizes ranging from 150 mm to 420 mm deep, up to 2400 mm wide, and up to 20000 mm long (Yang, 1994). Due to their design benefits, PHC slabs are usually used in repeated floors of industrial, residential and commercial buildings.

The unique shape of the PHC slab requires specific manufacturing methods. PHC slabs are manufactured using either the dry cast extrusion method or the wet slip form method. The process starts by cleaning and arranging the prestressing strands along the casting bed based on the designed prestressing forces for both casting methods. The first technique uses the extrusion machine, which shears compact dry low slump concrete while forming the empty cores with high speed moving tubes or augers. The other method is slip forming, which uses high slump concrete that is fed into the casting machine while the sides of the slab are formed by a stationary or a slip form (Palmer and Schultz, 2010). Because the extrusion method is more economical and
efficient, it is the preferred technique to produce PHC slabs in North America and globally. However, casting PHC slabs using the extrusion machine makes it difficult to add shear reinforcement inside the slabs. The absence of shear reinforcement in the prestressing transfer zone exposes the member leaving it unprotected against shear failure (Yang, 1994). Finally, PHC slabs are assembled side by side as simply supported slabs. Due to the inclined sides of PHC slab, a V notched space appears between the slabs. Concrete grout is used to fill the gap, and level the top of the slabs, as shown in Figure 1.1.

![Concrete Grout](image)

Figure 1.1 PHC slabs floor plan with filling

1.2 Motivation

Structures are designed to fail in ductile flexural failure mode in case of a sudden overloading. Prior to the event of a flexural failure, a structure member (such as beams or a one way slab) will experience large deflection and develops cracks in the tension zone of the member. This gives a warning prior to collapse. However, any concentrated load near the supports might result in a shear failure. Shear failure is known to be brittle in nature. It does not give any indications, such as an alert, in the event of a member collapse. The extrusion method by which PHC slabs are manufactured makes it difficult to arrange stirrups. Thus, the concrete tensile strength is the only element resisting the shear forces applied on the PHC slabs.
The shear capacity of the PHC slabs can be improved in various ways. The first method is to reduce the number of cores by filling several cores over the full slab length. The second solution is to break into the cores locally and fill over a specific length of the cores while the concrete is still in a plastic state (Buettner and Becker, 1998). Another approach to enhance the shear capacity is to increase the slab thickness. However, all these solutions will increase the self weight and the cost of the slabs, while increasing the slab thickness to more than 305 mm that will result in a reduction in the shear capacity due to the size effect (Palmer and Schultz, 2010). Therefore, it is essential to find a strengthening technique to increase the shear capacity of the PHC slabs without increasing the cost or the weight of the slabs.

For more than three decades, fibre reinforced polymers (FRP) have been a solution to repair or strengthen reinforced concrete members (Kansara et al., 2010). FRP sheets are a composite material that consists of longitudinal fibres embedded in a polymer matrix (Khalifa and Nanni, 2012). According to Alkhrdaji (2015), the fibres act as the primary reinforcement, while the polymer matrix, which is a resin, provides the protection for the fibres and transfers the load between fibres. Furthermore, FRP sheets are used as a strengthening technique for several reasons: (1) to enhance the structure carrying capacity in flexure and shear to sustain a safe structure or to satisfy specific serviceability requirements for heavy duty structures such as bridges; (2) to provide extra strength for under designed members or build with substandard materials; and (3) to restore the capacity of a deteriorated concrete infrastructure. Additionally, FRP sheets have three main fibres types: aramid (AFRP); carbon (CFRP); and glass (GFRP). FRP has many advantages over traditional methods of concrete rehabilitation and strengthening techniques. Primarily, FRP sheets are corrosion resistant, high fatigue resistant and stiff compared to their weight as well they are highly durable and flexible so that they can take any
FRP sheets can be used to increase the shear capacity with and without the presence of steel stirrups in various structural elements. Previous research was conducted to investigate the effectiveness of bonding FRP sheets externally to PHC slabs webs. The first phase of the study was done by Wu (2015) who investigated the feasibility of strengthening PHC single web beams with CFRP sheets. The studied parameters included the prestressing levels, the CFRP sheets’ length as well as the number of layers. Results showed that using CFRP sheets enhanced the shear capacity of the webs’ beam by up to 44% over the control specimen. For Phase II of the research, full width PHC slabs were strengthened by CFRP sheets for shear by Meng (2016). Phase II of the research investigated the same parametric effects as in Phase I. Strengthening full width PHC slabs with CFRP sheets showed excellent results in enhancing the shear capacity by up to 38%, and the ductile behaviour of the strengthened slabs was greater than the control specimen.

According to Alferjani et al. (2013), CFRP sheets have no significant difference in performance over GFRP sheets. However, GFRP sheets are known to be cheaper than CFRP sheets. Therefore, it is important to consider GFRP sheets as an alternative to CFRP sheets while strengthening PHC slabs in shear. In this research, the inner surfaces of PHC slabs webs will be strengthened using GFRP sheets to enhance the shear capacity of the slabs.
1.3 Objectives
The objectives of the current study proposed are as follows:

1. Investigate the effectiveness of strengthening full width PHC slabs using GFRP sheets on the shear capacity.

2. Conduct a parametric study to evaluate the effectiveness of the strengthening technique using GFRP sheets of different arc widths and different installation methods on the shear capacity of different prestressed level slabs.

3. Compare the behaviour of the GFRP strengthened PHC slabs with the CFRP strengthened ones.

4. Compare experimental results to that of the codes of the American Concrete Institute (ACI) and Canadian Standards Association (CSA) and their predictions.
 CHAPTER 2: LITERATURE REVIEW

This chapter reviews previous research of all the composite parts involved in the current research. The main component of the current research is the PHC slab. Therefore, all failure mechanisms of PHC slabs and the factors affecting web shear failure in PHC slabs are stated. In addition, PHC slabs are designed as a one way slab system. According to ACI 318-14 (2014), one way slabs are designed like beams, therefore previous research on the shear strengthening of beams using FRP sheets are included in this review. Furthermore, two studies of strengthening single web and full width shear strengthening of PHC slabs using externally bonded CFRP sheets in shear are also included. Finally, the American Concrete Institute’s (ACI) and Canadian Standards Association’s (CSA) codes, provisions and guidelines utilized to predict the shear capacity of PHC slabs are also included.

2.1 Web Shear Strength of PHC Slabs

PHC slabs are simply supported one way slabs with no transverse reinforcement. They are not subjected to any stresses caused by a negative bending event or a torsion event. Yet, PHC slabs are subjected to positive bending moments and shear stress from the applied load and local tensile stress caused by the prestressing forces. Various PHC slab failure mechanisms are categorized under the limit state design (LSD).

2.1.1 Limit state design (LSD)

Limit state design (LSD) is a structural engineering design method. Limit state design is a term used to describe structures that cannot fulfill their design requirements and become unfit for their intended use such as: fitness for use; structural integrality; fire resistance; and durability. The LSD method includes two main design criteria: the ultimate limit state (ULS); and the serviceability limit state (SLS).
2.1.1.1 Ultimate limit state (ULS)

Ultimate limit state (ULS) involves the structural collapse of a full structure or part of it. Such a limited state should have a very low probability of occurrence as it might lead to loss of life and major financial losses. The ultimate limit state consists of the following: (1) loss of equilibrium of a part or all of a structure’s members; (2) rupture of critical part of the structure; and (3) progressive collapse (Wight and MacGregor, 2012). Some of the PHC slab failure modes under ULS are shown in Figure 2.1.

2.1.1.2 Serviceability Limit State (SLS)

Serviceability limit state (SLS) is the design requirement for structures during its service life. The criteria for the SLS involves the disruption of the functional use of the structure and the probability of occurrence that can be generally tolerated at an ultimate limit state. All structural members must be designed to satisfy: deflection control; excessive crack width; and undesirable vibrations requirements (Wight and. In addition, structural members must meet the minimum requirement of reinforcement area and spacing. Finally, the design should fulfill having vibrations in acceptable limits for the intended use. The failure modes for SLS are summarized in Figure 2.2.
2.1.2 Shear tension and compression failure in web

When the tensile strength at the beam’s bottom flange is high enough, and the concrete does not crack in flexure, an inclined crack may occur in the web near the support where the development length of the prestressed strands is too short to prevent the failure (Yang, 1994). The shear failure is predicted to happen when the principal tensile stress at the web resulted from shear and when bending exceeds the concrete tensile strength. The diagonal crack spreads both up to the top and down to the bottom, which results in a brittle failure as shown in Figure 2.1(e) (Pajari, 1991).

The shear compression failure happens when compressive stress in the web exceeds the maximum concrete compressive strength of $3500 \mu \varepsilon$ (CSA-A23.3-14, 2014). In addition, failure occurs after an existing crack width increases with the load until the concrete at the top crashes down (Pajari, 1991). However, without shear reinforcement, compression shear failure cannot happen as the tension shear failure will occur first since the concrete tensile strength is less than the concrete compressive strength (Yang, 1994).

2.1.3 Factors affecting web shear capacity

PHC slabs have no shear reinforcement, which increases the risk of brittle shear failure once an inclined crack is formed. The shear capacity of PHC slabs is affected by six different parameters that are listed below.
2.1.3.1 Depth of the member

The ACI 318-14 (2014) shear equation (22.5.8.3.2) states that the tensile strength of the concrete is the only contributor to the shear carrying capacity of a reinforced concrete member without shear reinforcement. In addition, any increase in the depth of the beam of more than 305 mm with very little (or no) shear reinforcement will decrease the shear capacity. Palmer and Schultz (2010) stated that the size effect becomes critical in depths greater than 510 mm. Wight and MacGregor (2012) found that increasing the thickness of the member will lead to an increase in crack width. Thus, the maximum shear stress transferred by aggregate interlock is reduced with the result of a decrease in the shear capacity. To date, there is no research on the size effect on PHC slabs.

2.1.3.2 Geometry of the cross section

The geometry of PHC slabs is categorized into two types: circular and non-circular cores. The differences in the cross sectional geometry would affect the shear strength. The location of the shear critical point varies from one shape to another. Yang (1994) found that the shear critical point in the circular voids of a PHC slab was at the intersection of the centre of the slab with a line of 35° inclination away from the face of the support, as shown in Figure 2.3. However, the shear critical point for noncircular voids was located at the intersection of the narrowest point in the bottom flange web and an inclined line with an angle of 35°. Another factor is the transverse distribution effect, which assumes that webs carry even and uniform precompressive forces. However, design variability or the manufacturing process results in an uneven shear distribution that reduces the shear capacity significantly to the web with the least thickness (Palmer and Schultz, 2010).
2.1.3.3 Axial stress due to prestressing forces

The web tension shear failure occurs when the principal tensile stress in concrete exceeds its tensile strength. The presence of axial tensile force decreases the shear capacity of the member. In addition, the axial tensile force increases the tensile strain in the longitudinal reinforcement, which will lead to an increase in the crack’s width. The axial compressive force increases the shear capacity of PHC slabs by reducing the principal tensile stress of the member produced by the prestressing force. Furthermore, the prestressing compression force reduces the longitudinal strain, which results in reducing the crack width (Wight and MacGregor, 2012).

2.1.3.4 Tensile strength of concrete

Unreinforced concrete sections tend to fail in shear when the maximum principal tensile stress reaches the concrete tensile strength. According to Pisanty (1992), the tensile strength of the concrete varied from the bottom to the top of the slab as an outcome of the casting process, compaction and curing. Therefore, there is no way to predict an accurate value of the concrete’s tensile strength. In addition, the stresses in the PHC slab webs consists of biaxial principal tensile and compressive stress instead of a uniaxial stress, which reduces the tensile strength of the concrete (Wight and MacGregor, 2012).
2.1.3.5 Shear lag of prestressing force

Shear lag is a phenomenon where the compressive stress near the support and just above the strands is less than the compressive stress around the strand. Correspondingly, the compressive stress resulted from the prestressed strands is spread into the section with a 45° angle. The shear lag phenomenon is known to reduce the shear capacity as the precompressive stress by the strands does not contribute to resisting the shear stress. However, the shear lag effect is ignored in the ACI 318-14 (2014) formula (22.5.8.3.2) because the critical point is assumed to be outside the affected zone (Palmer and Schultz, 2010).

2.1.3.6 Shear span to depth ratio (a/d)

The span to depth ratio (a/d) affects the shear capacity of the member based on the location of the load from the supporting point and the thickness of the member, as shown in Figure 2.4. The thickness of the member remains constant throughout the span. Therefore, the flexural capacity remains the same, while the shear capacity varies based on the loading location. The shear span to depth ratio is divided into many sectors. The first is the deep beam sector, which has an a/d range of 0 to 1. Members in the deep beam sector usually fail due to an anchorage failure at the end of the tension tie. The second is the short shear span, which covers a/d from 1 to 2.5. The failure mode starts by an inclined crack followed by a bond failure, a splitting failure or a dowel failure along the tension reinforcement. The member also might crush in the compression zone over the top of the crack and lead to shear compression failure. Third is the slender shear span with an a/d from 2.5 to 6. This occurs when the member fails with an inclined crack with respect to the loading direction. The fourth occurs when the very slender beams, which have an a/d ratio of 6 and more, fail in flexure (Wight and MacGregor, 2012).
2.2 Previous Work Using FRP Sheets to Externally Strengthen Concrete Beams and PHC Slabs

It has been proven that FRP increases the shear capacity of reinforced concrete (RC) structures. FRP application varies based on: the type of the fibre composite; fibre orientation; surface area covered; thickness of the FRP sheets; and the continuity of the FRP sheets through the member length.

2.2.1 Shear strengthening of RC beams using CFRP Sheets

Li et al. (2001) conducted experiments on RC beams strengthened with unidirectional CFRP sheets to investigate the stress distribution, crack propagation, and the ultimate strength. The tests were conducted on five different beams with different CFRP sheet configurations, as shown in Figure 2.3. The beam setup was based on four-point loading. The first specimen was only strengthened in bending. The second to the fifth specimens were strengthened in both bending and shear for different depths. The results demonstrated that covering the whole concrete surface area was not necessary. Additionally, the results illustrated that to strengthen beams in shear, the beam had to be strengthened in flexure too, as the first crack appeared in the
concrete tension zone. Strengthening RC beams in flexure would delay the formation of the first crack and the shear crack.

![Diagram of beam cracks after failure](image)

**Figure 2.5 Beam cracks after failure (Li et al., 2001)**

Adhikary and Mutsuyshi (2004) tested eight simply supported RC beams strengthened with unidirectional CFRP sheets to investigate the effects of different CFRP sheets configurations and layouts on the ultimate shear strength of the beams. There were two different wrapping schemes: U wrapping, and two sides of the beam. The CFRP sheets also had different alignments (90°, 0° and 90° + 0° to the beam’s longitudinal axis), as shown in Figure 2.6. Results showed that the shear capacity of the beam increased when the number of CFRP sheets was increased. In addition, vertical U wrapping was found to be the most effective wrapping
scheme to increase the beam shear capacity, with a maximum increase of 119% over the control specimen.

Figure 2.6 CFRP sheets bonding layouts for all beams (Adhikary and Mutsuyshi, 2004)

Jayaprakash et al. (2008) investigated the shear capacity of precracked and uncracked beams without any internal reinforcement and strengthened with bidirectional CFRP strips. The testing program consisted of 16 beams: four control beams, six uncracked beams strengthened with CFRP strips and six precracked beams strengthened with CFRP strips. The variables examined were the: longitudinal tensile reinforcement ratio; the shear span to effective depth ratio and the spacing of CFRP strips; and the orientation of the CFRP strips. All specimens were tested using the four points bending test. The shear capacity increased between 11% and 139%
over the control beam. The results showed that increasing the tensile strength of concrete, CFRP strips spacing and the orientation of the strips were found to affect both the cracking pattern and the shear capacity.

2.2.2 Shear strengthening of RC beams using GFRP sheets

Saffan (2006) investigated the efficiency of using GFRP wraps to strengthen reinforced concrete beams with insufficient shear reinforcement. A total of 18 specimens were tested. Twelve specimens were control beams, which varied between four combinations of four parameters: (1) no web reinforcement; (2) with steel web reinforcement; and (3) with/without FRP flexural strengthening. The rest of the beams had different GFRP warp configurations like side wraps, U jacket wrapping and full wrapping. All specimens were tested in four point bending of a 300 mm span and an a/d of 2.3. The results demonstrated that strengthening beams with GFRP could lead to a significant increase in the beam’s shear capacity. In addition, it was noticed that the U jacked wrap could alter the failure mode from brittle shear failure to ductile flexural failure.

Sundarraja and Rajamohan (2009) carried out an experimental study to clarify the effect of installing strips on beams’ web as shear reinforcement. The testing parameters were the width and the spacing of the inclined GFRP strips, the spacing of stirrups and the amount of longitudinal reinforcement. The experiments were performed on 13 different beams consisting of five different designs. The tests were conducted under two point loading method. The results showed that the ACI code predictions were accurate. GFRP strips also showed up to 50% enhancement in the ultimate shear capacity. A comparison of the findings presented by Sundarraja and Rajamohan (2009) with previous research in the same topic suggested that U wrapping had increased the ultimate shear capacity more than the inclined strips.
Chajes et al. (1995) tested twelve simply supported beams in a four point loading test to determine the increment in shear capacity resulted from adding bidirectional aramid, glass and graphite fibre reinforced U wraps to the beam. The specimen design lacked the minimum amount of shear reinforcement required and forced the beams to fail in shear. The initial crack was formed at the tension zone of the beam, and the FRP sheets did not debond prior to the failure. However, both the glass and graphite wraps were torn along the diagonal crack. All specimens failed in shear as designed, and the U wrapping was proven to be an effective method to increase the shear capacity of the beams.

Baggio et al. (2014) investigated shear strengthening of slender beams using different Sika Inc. products, CFRP, GFRP and FRCM wraps, and the presence of different types of anchorage. The testing was on a total of nine beams using the four point loading testing method. The results showed an increment of the shear capacity while using GFRP wrap with and without anchorage to be equal to 50% and 36%, respectively, and 34% and 75% for CFRP and FRCM wraps, respectively. Finally, the presence of anchorage was able to prevent the FRP sheets from debonding.

2.2.3 Summary

The previous research investigated different parameters affecting the shear capacity of RC beams strengthened externally by FRP sheets in shear. The different parameters are as follows:

1. Depth of FRP sheets

   FRP sheets/strips along the whole depth of beams concrete surface showed a close shear enhancement as beams that were 50% covered by FRP sheets.

2. Bonding types of FRP sheets
The bonding types of FRP sheets are full wrapped, U wrapping and side bonding. In most cases, full wrapping is impossible. Consequently, the best and most efficient type of FRP sheet layout configuration is vertical U wrapping.

3. Quantity of FRP sheets

The most common number of layers was either one or two layers. The studies showed that increasing the number of layers or the thickness of the FRP sheets would increase the load carrying.

2.2.4 Shear strengthening of PHC slabs using externally bonded FRP sheets

There is currently no existing research on the application of GFRP sheets to strengthen PHC slabs in shear. However, there is a study on the use of CFRP sheets with respect to strengthening single web PHC slabs (Phase I) and full width PHC slabs (Phase II). Phase I was conducted by a previous student at the University of Windsor (Wu, 2015). As shown in Figure 2.7, the study focused on single web beams cut longitudinally across the full width PHC slabs. The testing parameters were the number of CFRP layers, different lengths of the CFRP sheets and different prestressing levels. The objective of the research was to study the effectiveness and possibility of strengthening PHC slabs with CFRP sheets to increase the shear capacity.

Wu (2015) tested sixteen I-shaped single web beam specimens of the two series. The first series consisted of eight low prestressed I-beams (one prestressing strand per web) specimens. The second series included eight medium prestressed I-beams (two prestressing strands per web) specimens. All specimens were tested by a three point load flexural test for a span of 4499 mm where the a/d is 2 for the first series’ specimens and 2.5 for the second series’ specimens. The tested parameters included different CFRP sheets lengths: 300 mm, 450 mm and 600 mm, as well as different number of CFRP layers: one and two.
The results of Phase I show an enhancement in the shear capacity and ductility of the specimens prior to failure. After trying different number of layers, the most feasible configuration was after adding two layers of CFRP with an average shear increment of 14.5% for the first series and 27.25% for the second series over the control specimens (Wu, 2015).

In Phase II, the research was extended to conduct experiments on full width PHC slabs. Meng (2016) followed the same testing parameters in Phase I by Wu (2015). The study had two types of slabs: low prestressed profile PHC slabs (one prestressing strand per web – S1-Series), and medium prestressed PHC slabs (two prestressing strands per web – S2-Series), which were tested from both sides to give a total of nine specimens. The CFRP sheets were installed on all the internal webs of the prestressed concrete, as shown in Figure 2.8. As in Phase I, all specimens were tested based on a three point load flexure test and a span of 4449 mm and loaded with an a/d of 2.5. The results for the first and the second series showed an average increase of the shear capacity by 16.33 % and 26.4% over the control specimen, respectively.

Figure 2.7 Shear strengthening techniques for single web hollow core slabs (Wu, 2015)
2.3 Code Provisions and Guidelines for Shear Capacity of PHC Slabs

2.3.1 ACI-318-14 code

The ACI-318-14 code (2014) limits the ultimate shear capacity \( V_u \) to be the lesser of the flexure shear cracking \( V_{cl} \) and the web shear cracking \( V_{cw} \). The ACI-318-14 code does not require shear reinforcement in one way slabs PHC slabs if the ultimate shear does not exceed \( 0.5\phi V_c \) for slabs with thickness larger than 254 mm (10 inches) or more. For slabs with thickness less than or equals to 254 mm (10 inches), the ACI code assumes that concrete would contribute to the shear capacity through concrete compressive zone, aggregate interlock and dowel action.

To calculate the nominal web shear capacity \( V_{cw} \), the equation in ACI-318-14 code (2014) is (22.5.8.3.2):

\[
V_{cw} = (3.5\sqrt{f'_c} + 0.3f_{pc})b_wd_p + V_p \quad \text{(Imperial)}
\]

or

\[
V_{cw} = (0.29\sqrt{f'_c} + 0.3f_{pc})b_wd_p + V_p \quad \text{(metric)}
\]

where the specified compressive strength of concrete is noted as \( f'_c \). The web width and the distance from extreme compression fibre to the centroid of prestressing steel are noted as \( b_w \) and \( d_p \), respectively. The vertical component of effective prestressing force at critical section \( V_p \) equals zero for the PHC slabs because the strands are not draped or harped.

Finally, equations
(2.1.1) and (2.1.2) calculate the shear capacity based on the averaged shear stress distribution through the cross section.

### 2.3.2 CSA-A23.3-14

Canadian Standards Association (2014) adopted the Modified Compression Field Theory, which considers the post cracking shear strength of the member while estimating the shear strength of the member. The shear capacity carried by concrete is estimated by equations (11-11) to (11-14) in CSA A23.3-14 as follows:

\[
V_c = \Phi_c \lambda \beta \sqrt{f'_c b_w d_v}
\]  
\[\beta = \frac{0.4}{1 + 1500 \varepsilon_x} \left[ \frac{1300}{1000 + S_{ze}} \right]
\]  
\[
\varepsilon_x = \frac{M_f / d_v + (V_f - V_p - 0.5 N_f - A_p f_{po})}{2(E_s A_s + E_p A_p)}
\]  
\[
S_{ze} = \frac{35 s_z}{15 + a_g} \geq 0.85 s_z
\]

where the concrete strength reduction factor \(\Phi_c\) is 0.65. The effective web width \(b_w\) is equal to the minimum concrete web width within the depth \(d\). The effective shear depth \(d_v\) is equal to the largest of 0.90 \(d\) (depth from top fibres until the middle of the longitudinal reinforcement) and 0.72 \(h\) (full depth). The angle of inclination of the principal diagonal compressive stress to the longitudinal axis, and the ability of cracked concrete to transmit shear by aggregate interlock are noted as \(\theta\) and \(\beta\), respectively. The effective crack spacing and the longitudinal strain at mid-depth of the section are noted as \(S_{ze}\) and \(\varepsilon_x\), respectively. The crack spacing parameter \(s_z\) can be obtained from the longitudinal spacing between transverse cracks at mid depth is dependent on the longitudinal reinforcement.
2.4 Code Provisions and Guidelines for RC Members Strengthened in Shear using FRP

2.4.1 ACI 440. 2R-15

The ACI 440.2R-15 specifies that the design shear strength for concrete member strengthened with FRP sheets shall not exceed the required shear strength. Then, the shear capacity gained from the FRP sheets is added to the shear capacity of the concrete. Additionally, a factor $\Psi$ of 0.85 is added for members strengthened on two sides of a web. FRP capacity depends on the fibre orientation and assumed crack pattern. The shear contribution to RC members is given by Equation (11-3) in the ACI code:

$$V_f = \frac{2nt_fw_fE_f\varepsilon_{fe}(\cos\alpha_f + \sin\alpha_f)d_f}{S_f}$$

(2.3.1)

where the number of FRP layers, the nominal thickness of a single FRP sheet layer, the width of the FRP sheet, and the tensile modulus of elasticity of FRP are noted as $n$, $t_f$, $w_f$, and $E_f$, respectively. The effective strain level of the externally bonded FRP sheets at failure ($\varepsilon_{fe}$) should not exceed 0.004 for bonded face piles. The effective depth of FRP sheets and the spacing between the sheets are noted as $d_f$ and $S_f$, respectively.

2.4.2 CSA-S806-12

The Canadian Standards Association (2012) assumes FRP contribution to a retrofitted beam will be determined by Equation (11-2) in the code as follows:

$$V_f = \frac{\Phi_fA_FE_F\varepsilon_Fd_v(cot\theta + cot\alpha_f)sin\alpha_f}{S_f}$$

(2.4.1)

where the material resistance factor, cross sectional area, modulus of elasticity, effective strain, effective depth, and spacing of the FRP shear reinforcement are noted as $\Phi_f$, $A_F$, $E_F$, $\varepsilon_F$, $d_v$ and $S_f$, respectively. The orientation angle of the fibre with respect to the longitudinal angle is noted as $\alpha_f$. 
CHAPTER 3: EXPERIMENTAL PROGRAM

3.1 Test Specimens

A total of 11 full size PHC slabs specimens were tested in this study. All specimens had a length of 4500 mm, a depth of 305 mm and a width of 1219 mm. The test specimens were divided into three series, S1-Series, S2-Series and S3-Series, based on the prestressing strand profile, i.e., the prestressing level. The S1-Series (low prestressing of 8.35 MPa) contained one specimen that has six 13 mm diameter prestressed strands at the bottom side of the slab. The four specimens in the S2-Series had eight 13-mm prestressing strands at the bottom side, giving a medium prestressing of 11.13 MPa. There were six specimens in the S3-Series, which had a bottom strand profile identical to those in the S2-Series, i.e., eight 13 mm prestressing strands, in addition to two 10 mm top prestressing strands, each is 13 mm in diameter, which resulted in a prestressing level of 13.91 MPa. Figure 3.1 shows the details of the specimens of each series.

Table 3.1 presents the details of the test matrix. For the S3-Series, test specimen S3-C was used as a control specimen, with no GFRP sheets applied where “S3” refers to a specimen with high prestressing level, and “C” represents the control slab. For all of the strengthened slabs, two layers of GFRP sheets for 450 mm long in the longitudinal span direction were bonded on the internal surface of all the slab voids, as shown in Figure 3.2. GFRP sheets with an arc width equivalent to 120°, 150° and 180° were applied, as shown in Figure 3.3. Furthermore, two different installation procedures, the immediate wet installation method “W” and the precured installation method “P,” were followed. Accordingly, S3-2-450-180-W GFRP means a S3-Series PHC slab specimen “high prestressing level” was strengthened with two layers of 450mm long GFRP sheets on each side of the slab void surface over an arc width of 180° and applied using the immediate wet installation method.
a) S1-Series (low prestressing) slab (units: mm)

b) S2-Series (medium prestressing) slab (units: mm)

c) S3-Series (high prestressing with compression prestressing) slab (units: mm)

Figure 3.1 Cross section of slabs with different prestressing levels

Figure 3.2 Schematics of GFRP installation position (units: mm)
Figure 3.3 Cross section of different GFRP sheets arc lengths

Table 3.1 Testing matrix

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>No. of GFRP Layers</th>
<th>Length (mm)</th>
<th>Arc Width (°)</th>
<th>Installation Method</th>
<th>Bonding Epoxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S1-2-450-180-W</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>S2-2-450-180-W</td>
<td>2</td>
<td>450</td>
<td>180</td>
<td>Wet</td>
<td>Sikadur 300</td>
</tr>
<tr>
<td>3</td>
<td>S2-2-450-180-W 2nd</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>S2-2-450-150-W</td>
<td></td>
<td></td>
<td>150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>S2-2-450-120-W</td>
<td></td>
<td></td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>S3-C</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>S3-2-450-180-W</td>
<td></td>
<td></td>
<td>180</td>
<td>Wet</td>
<td>Sikadur 300</td>
</tr>
<tr>
<td>8</td>
<td>S3-2-450-150-W</td>
<td></td>
<td></td>
<td>150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>S3-2-450-120-W</td>
<td></td>
<td></td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>S3-2-450-150-P</td>
<td></td>
<td></td>
<td>150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>S3-2-450-120-P</td>
<td></td>
<td></td>
<td>120</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.2 Preparation of Test Specimens

All specimens were precast and cured at Prestressed Systems Inc. (Windsor, ON), using the extrusion method. Then, the specimens were shipped to the Structural Lab at the University of Windsor for testing. Before strengthening the PHC slabs, the surfaces of the cores were polished using a steel brush, and all dust was removed by compressed air. There are two installation methods in Phase III.

3.2.1 Wet installation method

The first method used to put on the GFRP sheets was the immediate wet installation method, which starts by saturating the web surfaces and GFRP sheets with epoxy resin. Immediately, the saturated GFRP unidirectional sheets are bonded to the void surface parallel to the vertical axis of the PHC slab, as shown in Figure 3.4. The PHC slab is tested after the GFRP sheets were cured for two weeks. A step by step demonstration of the wet installation method was included in the appendices.

3.2.2 Precured installation method

The second installation method starts by curing GFRP sheets to take an arc shape that matches the web surface geometry for 24 hours. Then, a dry paste, Sikadur 330, is placed on the void surface and one cured GFRP unidirectional sheet are bonded to the void surface parallel to the vertical axis of the PHC slab, as shown in Figure 3.4. Lastly, another layer of the dry paste is placed and a second GFRP layer is attached on top of the paste. A step by step demonstration of the precured installation method was included in the appendices.
3.3 Material Properties

3.3.1 Concrete

The concrete used in casting the test specimens was a normal weight concrete with an average compressive strength of 55 MPa. In addition, the modulus of elasticity \( E_c \) was in the range of 22000 MPa and 24000 MPa based on the experimental testing done in Phase II (Meng, 2016).

3.3.2 Epoxy

For the wet installation method, Sikadur 300 epoxy was used for impregnation and as a bonding agent. Sikadur 300 is a type of two component polymer epoxy, with a mixing ratio of 2.38 to 1 by volume. Sikadur 300 has a tensile modulus of 1.724 GPa and a tensile strength of 55 MPa. The viscosity of the epoxy is approximately 500 cps, applicable to serve in a temperature range of -40° to 60° C, and has a tack free time of up to 16 hours (Sika Canada Inc., 2016a).

For the dry installation method, Sikadur 330 epoxy was used as a bonding agent between the cured GFRP sheets and the concrete. Sikadur 330 is a two component solvent free epoxy, with a mixing ratio of 4 to 1. In addition, Sikadur 330 has many advantages such as: long pot life; easy to mix; solvent-free; high temperature; and creep resistance. The density, tensile strength, and flexural modulus of elasticity are 1.31 kg/L, 30 MPa and 38 GPa, respectively. The
tack free time for Sikadur 330 is 56 minutes and is applied to serve in a temperature range between -40° to 50° C (Sika Canada Inc., 2016b).

3.3.3 GFRP sheets

The GFRP sheets used in the experimental work are one of Sika’s products as well. It is called SikaWrap Hex 100G, which is an unidirectional glass fibre, and its features include it being a lightweight, noncorrosive, acid resistant, low aesthetic impact and economical wrap. Sika Hex 100G wrap has a tensile strength of 2,276 MPa, a tensile modulus of 72,413 MPa, 4% elongation, density of 2.54 g/cc and a nominal thickness of 0.359 mm (Sika Canada Inc., 2016c).

3.3.4 Prestressing strands

The prestressing strands used are 7 wires with an overall diameter of 10 mm or 13 mm. The strands are low relaxation and have an ultimate tensile strength of 1860 MPa.

3.4 Test Setup and Instrumentations

The testing was carried out in the Structural laboratory at the University of Windsor. All specimens were setup based on the three point flexure testing procedure and instrumented at the loaded end, as shown in Figure 3.7. The slab was subjected to a constant loading rate of 10 kN/min until failure. Two strain gauges were installed to measure the GFRP strain at the mid height of the edge web and mid web. Four Pi gauges were installed to measure the concrete strain. The first and the third Pi gauges measured the concrete strain in the longitudinal direction of the slab at mid height of edge webs. Furthermore, the second Pi gauge measured the concrete strain the same as Pi gauges one and three but at mid web, as shown in Figure 3.6. Wu’s (2015) results showed that the best position to place strain gauges and pi gauges were at the intersection of the horizontal centre line and a line connecting the inner face of the support to the inner face of the loading beam. The fourth Pi gauge was installed at the top surface of the slab near the
loading point to measure the concrete compressive strain at that location. Three linear variable
displacement transducers (LVDT) were used: two LVDTs to measure the prestressed strands
slippage at an edge web and a mid web, and the third LVDT to measure the slab vertical
deflection near the loading point, as shown in Figure 3.5 to Figure 3.8.

Figure 3.5 Overall experimental setup at loaded end
Figure 3.6 Side view of the test setup (units: mm)

Figure 3.7 The experiment full test setup
Figure 3.8 Cross section view of the test setup (units: mm)
CHAPTER 4: TEST RESULTS AND ANALYSIS

This chapter is divided into several subsections. First, sections 4.1, 4.2, 4.3 and 4.4 present a parametric study of the effect of the investigated test parameters on the behaviour and ultimate shear strength of the strengthened specimens. The investigated test parameters included:

1. Strengthened GFRP sheets width;

2. Prestressing levels;

3. Installation method of the GFRP sheets; and

4. Type of the FRP strengthening sheets (GFRP versus CFRP).

In each section, the test results are presented in terms of comparisons of ultimate loading capacity, failure mode, the load deflection relationship at the loading position, cracking behaviour, load strand slippage relationship and strain behaviour. Finally, Section 4.5 presents the cost analysis of strengthening PHC slabs by applying both the CFRP and GFRP sheets using both the wet and dry installation methods.

4.1 Effect of Width of the Strengthening GFRP Sheets

In the current research, four S2-Series specimens with two 13 mm prestressing strands in each web were tested. All specimens were strengthened with two layers of GFRP sheets for 450 mm long using the wet installation method. Two specimens were strengthened over 180° arc width (S2-2-450-180-W GFRP and S2-2-450-180-W GFRP 2nd), one specimen over 150° arc width (S2-2-450-150-W GFRP), and another over 120° arc width (S2-2-450-120-W GFRP). All specimens are compared to specimen S2-C, which is the S2-Series control specimen tested in Phase II of the research program (Meng, 2016). The details of the five test specimens are summarized in Table 4.1.
Table 4.1 S2-Series specimens’ parameters

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Arc Width (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-C</td>
<td>-</td>
</tr>
<tr>
<td>S2-2-450-180-W GFRP</td>
<td>180</td>
</tr>
<tr>
<td>S2-2-450-180-W GFRP 2nd</td>
<td>180</td>
</tr>
<tr>
<td>S2-2-450-150-W GFRP</td>
<td>150</td>
</tr>
<tr>
<td>S2-2-450-120-W GFRP</td>
<td>120</td>
</tr>
</tbody>
</table>

Notes. All specimens except the control specimen were strengthened by GFRP sheets using the wet installation method. All specimens were medium prestressing level slabs (11.13 MPa).

4.1.1 Test observations

During the testing process, specimen S2-C failed due to a web shear failure. The shear crack developed at 135 mm from the face of the slab with an angle of 39° with respect to the longitudinal axis of the slab at a maximum load of 280.5 kN. Specimen S2-2-450-180-W GFRP failed at a load equal to 380 kN due to a typical shear failure. This shear crack was located at 90 mm from the face of the support and propagated with an angle of 36° with respect to the longitudinal axis of the slab. A replicated specimen, S2-2-450-180-W GFRP 2nd, developed its shear crack at a peak load equal to 434 kN and was 100 mm away from the face of the slab with an angle of 47° with respect to the longitudinal axis of the slab. A flexural crack at 1050 mm away from the face of the slab’s loaded end appeared when the applied load reached 380 kN prior to the formation of the shear crack. Specimen S2-2-450-150-W GFRP had a vertical flexural crack 770 mm away from the face of the slab at a load equal to 280 kN followed by a typical web shear failure once the applied load reached 425 kN. The shear crack was located at 300 mm with an angle of 45° with respect to the slab’s longitudinal axis. Finally, specimen S2-2-450-120-W GFRP developed a flexural crack at a load of 290 kN followed by shear failure at a load of 443 kN. The shear crack was 200 mm away from the face of the slab’s loaded end and
grew upward with an angle of 40° with respect to the longitudinal axis of the slab. The crack patterns of the S2-Series specimens are plotted in Figure 4.1.

Figure 4.1 S2-Series crack profile based on different strengthening width (units: mm)

After failure, it was possible to peel only the GFRP sheets with 150° and 120° arc widths off the concrete surface, as shown in Figures 4.2 and 4.3, respectively. The traces of concrete attached to the GFRP sheets indicate excellent bond between the GFRP sheets and the concrete up to failure.

Figure 4.2 Concrete traces on two 150° arc width GFRP sheets after manually peeling them off the PHC slab web
Figure 4.3 Concrete traces on two 120° arc width GFRP sheets after manually peeling them off concrete

4.1.2 Ultimate load

The test results of the five specimens are all summarized in Table 4.2. All specimens failed in a brittle tension shear failure mode. Specimens S2-C, S2-2-450-180-W GFRP, S2-2-450-180-W GFRP 2nd, S2-2-450-150-W GFRP, and S2-2-450-120-W GFRP failed at 280.5 kN, 380 kN, 434 kN, 425 kN, and 443 kN, respectively. The results suggest that changing the GFRP sheets arc width has a limited effect on enhancing the ultimate shear capacity of the GFRP sheets strengthened specimens.

Table 4.2 Test results of S2-Series slab specimens based on different strengthened widths

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Failure Load (kN)</th>
<th>Percentage of Improvement (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-C</td>
<td>280.5</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-180-W GFRP</td>
<td>380</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-180-W GFRP 2nd</td>
<td>434</td>
<td>54</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-150-W GFRP</td>
<td>425</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-120-W GFRP</td>
<td>443</td>
<td>58</td>
<td></td>
</tr>
</tbody>
</table>
4.1.3 Load deflection behaviour

Figure 4.4(a) shows the load deflection curve of the S2-Series specimens near the loading position. Initially, all specimens had the same linear behaviour prior to cracking. However, after cracking at 280 kN as in the S2-C case, each of the strengthened specimens showed different post cracking displacements and ultimate loads. Specimen S2-C failed suddenly after the development of a shear crack at a load of 280.5 kN and a 2.8 mm deflection. Specimen S2-2-450-180-W GFRP showed a reduced flexural stiffness after 340 kN, which was characterized by a rapid increase in deflection compared to the increase in the applied load. The slab failed at a peak load of 380 kN and a deflection of 5.5 mm. Specimen S2-2-450-180-W GFRP 2nd had a sudden reduction in the flexural stiffness once a flexural crack appeared. The slab failed at a load of 434 kN and a maximum deflection of 5.1 mm. Specimen S2-2-450-150-W GFRP showed the largest reduction in the flexural stiffness due to the growing flexural crack with a 25 mm deflection at peak load. The load deflection behaviour indicated that the GFRP sheets started to carry the shear stress after the concrete cracked at 280 kN, which resulted in increasing of the ultimate shear capacity up to 425 kN. Just like the S2-2-450-180-W GFRP 2nd specimen, specimen S2-2-450-120-W GFRP had a sudden decrease in the slab stiffness at a load of 390 kN, which was followed by an abrupt decrease when a load of 443 kN and 4.8 mm deflection were reached. In addition, the load-deflection behaviour showed that the GFRP sheets width had an insignificant effect on both the shear capacity enhancement and the deflection at the ultimate load.

Figure 4.5 shows the relationship between the strand end slippage at the loading. It can be seen that all PHC slabs had almost no slippage prior to the ultimate load. After the ultimate load was reached, the slippage increased rapidly.
Figure 4.4 Load versus deflection near loading point based on different GFRP strengthening widths in S2-Series specimens

a) Full range

b) Close-up
Figure 4.5 Load end slip behaviour based on different GFRP strengthening widths in S2-Series specimens
4.1.4 Strain behaviour

Figure 4.6 shows the relationship between the GFRP strain measured parallel to the fibre direction obtained from strain gauge 1 and the applied load. All specimens had a linear behaviour during the elastic range. The elastic phase was ended by the appearance of the first crack. Thereafter, the GFRP sheets started to contribute in carrying the shear stress, which caused a rapid increase in the GFRP strain until the slab failed due to a tension shear failure. The maximum recorded strain was about 625 $\mu$ε at a load of 365 kN. The results suggest that the role of GFRP sheets is similar to the one of steel stirrups.

Figure 4.6 Load versus longitudinal tensile strain in FRP sheets for S2-Series

Figure 4.6 Load versus longitudinal tensile strain in FRP sheets for S2-Series
4.2 Effect of Prestressing Level

Three levels of prestressing are compared in the present research. The investigated prestressing levels included: (1) low-prestressing, 8.35 MPa, as in specimen S1-2-450-180-W GFRP (with one 13 mm strand in each web); (2) medium prestressing level, 11.13 MPa as in specimen S2-2-450-180-W GFRP 2nd (with two 13 mm stands in each web); and (3) high prestressing level, 13.91 MPa, as in specimen S3-2-450-180-W GFRP (with two 13 mm bottom strands and one 10 mm top strand in each web). All three slabs were strengthened using 180° arc width and 450 mm length GFRP sheets by the wet installation method. Another two slabs representing, respectively, the medium and high prestressing levels but with a 150° arc width GFRP sheets are included in the comparison. Their purpose is to study the effect of the prestressing level on the ultimate strength and behaviour when a smaller area is strengthened with GFRP sheets. All specimens included in the comparison are summarized in Table 4.3.

Table 4.3 S1-Series, S2-Series and S3-Series specimens’ parameters

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Arc Width (°)</th>
<th>Prestressing Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-2-450-180-W GFRP</td>
<td>180</td>
<td>8.35</td>
</tr>
<tr>
<td>S2-2-450-180-W GFRP 2nd</td>
<td>180</td>
<td>11.12</td>
</tr>
<tr>
<td>S3-2-450-180-W GFRP</td>
<td>180</td>
<td>13.91</td>
</tr>
<tr>
<td>S2-2-450-150-W GFRP</td>
<td>150</td>
<td>11.12</td>
</tr>
<tr>
<td>S3-2-450-150-W GFRP</td>
<td>150</td>
<td>13.91</td>
</tr>
</tbody>
</table>

Note. All specimens were strengthened by GFRP sheets which were installed by the wet installation method.

4.2.1 Test observations

Specimen S1-2-450-180-W GFRP experienced a typical shear tension failure like the rest of the specimens when the load reached 329 kN. The crack initiated at a distance of 460 mm from the face of the support at the loading end and propagated upward with an angle of 50° with respect to the longitudinal axis of the slab. As mentioned earlier, specimen S2-2-450-180-W
GFRP 2nd failed due to a typical shear failure at a load of 434 kN. Finally, specimen S3-2-450-180-W GFRP failed due to typical shear failure, which started in the right edged web then extended to the rest of the web at a load of 325 kN. The crack in specimen S3-2-450-180-W GFRP started near the support at a distance equal to 150 mm from the face of the slab and propagated with an angle of 40°. The crack profiles of the strengthened specimens are shown in Figure 4.7.

As previously mentioned, specimen S2-2-450-150-W GFRP failed at 425 kN load and 26.7 mm deflection. Specimen S3-2-450-150-W GFRP had a flexural crack that initiated at a load of 365 kN. The flexural crack spread across the bottom part of the specimen and closed after the unloading due to the prestressing forces. Two GFRP sheets detached from the right edge core concrete surface at failure when a shear crack occurred at a load of 396 kN. The shear crack started 76 mm from the face of the slab and extended with an angle of 60°. The crack profiles of both strengthened specimens are shown in Figure 4.8.

Some cracks appeared around the prestressing strands and in top and bottom flanges after the brittle failure. The cracks around the strands are believed to be caused by the strands’ slippage due to the high prestressing forces carried by the pretensioned strands, as shown in Figure 4.9.
Figure 4.7 Crack profile of S1-Series, S2-Series and S3-Series specimens strengthened with 180° arc width GFRP sheets (units: mm)

Figure 4.8 Crack profile of S2-Series and S3-Series specimens strengthened with 150° arc width GFRP sheets (units: mm)
Figure 4.9 Cracking patterns of S1-Series, S2-Series and S3-Series around the prestressing strands at loading end
4.2.2 Ultimate load

Table 4.4 shows a comparison between three specimens, S1-2-450-180-W GFRP, S2-2-450-180-W GFRP 2nd and S3-2-450-180-W GFRP, which failed due to a brittle shear crack at loads equal to 329 kN, 434 kN and 325 kN, respectively.

Table 4.4 Ultimate loads of three strengthened specimens using 180° arc width GFRP sheets

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Failure Load (kN)</th>
<th>Percentage of Improvement (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-2-450-180-W GFRP</td>
<td>325</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-180-W GFRP 2nd</td>
<td>434</td>
<td>55</td>
<td>Brittle</td>
</tr>
<tr>
<td>S3-2-450-180-W GFRP</td>
<td>325</td>
<td>25</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.5 shows the failure loads, the percentage of shear capacity improvement, and the failure mode of specimens S2-2-450-150-W GFRP and S3-2-450-150-W GFRP. Both failed due to a shear crack at loads of 425 kN and 396 kN, respectively. The results show that two slabs with two different prestressing levels had the same percentage of improvement, 51%, compared to their control specimen (specimens S2-C and S3-C, respectively).

Table 4.5 Ultimate loads of two strengthened specimens using 150° arc width GFRP sheets

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Failure Load (kN)</th>
<th>Percentage of Improvement (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-2-450-150-W GFRP</td>
<td>425</td>
<td>51</td>
<td>Brittle</td>
</tr>
<tr>
<td>S3-2-450-150-W GFRP</td>
<td>396</td>
<td>51</td>
<td></td>
</tr>
</tbody>
</table>

The results confirm Yang’s (1994) finding that the prestressing forces have no effect on the shear capacity of the slab, and the concrete properties are the main factor affecting the slabs’ shear capacity.
4.2.3 Load deflection behaviour

Figure 4.10 shows the load deflection curves of three specimens, S1-2-450-180-W GFRP, S2-2-450-180-W GFRP 2nd and S3-2-450-180-W GFRP. It can be seen from the figure that all specimens with different prestressing levels had the same linear trend in the elastic phase, which was discontinued by the appearance of the first crack. Both specimens, S1-2-450-180-W GFRP and S3-2-450-180-W GFRP, had the same exact post crack behaviour after the first crack appeared at loads of 329 kN and 284 kN, respectively, but specimen S1-2-450-180-W GFRP had a longer post peak trend. Specimen S2-2-450-180-W GFRP 2nd showed a better shear enhancement over the low and high prestressing PHC slabs. Therefore, prestressing forces do not fundamentally affect the shear capacity of PHC slabs, and the main factor affecting the shear capacity enhancement is the concrete’s property.

Additionally, specimens S2-2-450-150-W GFRP and S3-2-450-150-W GFRP had the same linear trend in the elastic phase which was ended when specimen S2-2-450-150-W GFRP had a flexural crack at a load of 290 kN, and when specimen S3-2-450-150-W GFRP had a shear crack at a load of 396 kN. After the flexural crack, specimen S2-2-450-150-W GFRP curve stiffness was reduced until a shear crack appeared at a load of 425 kN. Specimens S2-2-450-150-W GFRP and S3-2-450-150-W GFRP had a deflection of 25.96 mm and 4.86 mm, respectively, at the peak load, as shown in Figure 4.11.
Figure 4.10 Effect of prestressing level on the load deflection relation of S1-Series, S2-Series and S3-Series specimens strengthened with 180° arc width GFRP sheets

Figure 4.11 Effect of prestressing level on the load deflection relation of S2-Series and S3-Series specimens strengthened with 150° arc width GFRP sheets
Figure 4.12 shows the slippage of the strands from the face of the slab web at the loading end. No slippage was found prior to the ultimate load. The strands’ slippage started rapidly only after the peak load was reached. Specimens S1-2-450-180-W GFRP, S2-2-450-180-W GFRP 2nd and S3-2-450-180-W GFRP had a maximum slippage of 12.5 mm, 4.5 mm and 2.8 mm, respectively. Figure 4.13 shows the maximum slippage in specimens S2-2-450-150-W GFRP and S3-2-450-150-W GFRP that were 12.3 mm and 4.56 mm, respectively. The test results showed that the strands’ slippage trend is associated with the maximum load and deflection reached.

![Figure 4.12: Effect of prestressing level on the load strands slippage relation of S1-Series, S2-Series and S3-Series specimens strengthened with 180° arc width GFRP sheets](image)

Figure 4.12 Effect of prestressing level on the load strands slippage relation of S1-Series, S2-Series and S3-Series specimens strengthened with 180° arc width GFRP sheets
Figure 4.13 Effect of prestressing level on the load strands slippage relation of S2-Series and S3-Series specimens strengthened with $150^\circ$ arc width GFRP sheets

4.3 Effect of Installation Method of the GFRP Sheets

Four S3-Series specimens were strengthened with both the wet and precured installation methods. Two high prestressing specimens were strengthened with two layers of GFRP sheets for a 450 mm length and $150^\circ$ arc width using the wet installation method in specimen S3-2-450-150-W GFRP and the precured installation method in specimen S3-2-450-150-P GFRP. Additionally, another two high prestressing specimens were strengthened with two 450 mm long and $120^\circ$ arc width GFRP sheets using the wet installation method in specimen S3-2-450-120-W GFRP and the precured installation method in specimen S3-2-450-120-P GFRP. The four specimens strengthened with GFRP sheets are compared to the high prestressing level control specimen, S3-C. All of the specimens’ parameters are summarized in Table 4.6.
Table 4.6 S3-Series specimens' parameters

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Arc Width (°)</th>
<th>Installation Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-C</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S3-2-450-150-W GFRP</td>
<td>150</td>
<td>Wet</td>
</tr>
<tr>
<td>S3-2-450-150-P GFRP</td>
<td>150</td>
<td>Precured</td>
</tr>
<tr>
<td>S3-2-450-120-W GFRP</td>
<td>120</td>
<td>Wet</td>
</tr>
<tr>
<td>S3-2-450-120-P GFRP</td>
<td>120</td>
<td>Precured</td>
</tr>
</tbody>
</table>

Note. All specimens were high prestressing level slabs (13.91 MPa). All specimens were strengthened using GFRP sheets except the control specimen.

4.3.1 Test observations

Specimen S3-C failed due to a typical shear tension failure at a load of 262 kN. The crack initiated at the bottom of the slab 240 mm from the face of the slab at the loaded end and spread up with an angle of 29° with respect to the longitudinal axis of the slab. As previously mentioned, specimen S3-2-450-150-W GFRP failed due to a typical shear tension failure at a load of 396 kN. The specimen strengthened using the precured method, S3-2-450-150-P GFRP, reached an ultimate load of 405 kN. Specimen S3-2-450-150-P GFRP had a shear crack that spread from the bottom to the top with an angle equal of 70° with respect to the longitudinal axis of the slab and a distance of 550 mm from the face of the slab at the loading end. Both crack patterns are shown in Figure 4.14(a).

As mentioned before, specimen S3-2-450-120-W GFRP failed due to a typical shear tension failure at a load of 386 kN. The second specimen strengthened using the precured method, S3-2-450-120-P GFRP, started to fail when several cracks appeared in the top and bottom flanges at a load of 270 kN. Then, a typical shear crack appeared in three out of the five webs at a load of 293 kN and propagated from the bottom of the slab at a distance of 490 mm.
from the face of the slab at the loaded end with an angle of 34° with respect to the longitudinal axis of the slab.

The results show that both specimens strengthened using the precured installation method had a shear crack that initiated outside the strengthened zone. Additionally, the two different installation methods did not change the mode of failure for all the tested specimens.

Figure 4.14 Crack profile of S3-Series specimens based on the installation method (units: mm)

a) 150° arc width

b) 120° arc width
Figure 4.15 shows the difference between the bonding established using the wet installation method and the precured installation method. Unfortunately, only one 150° arc width GFRP sheet installed using the wet installation method was peeled off. Therefore, it was only possible to compare the contact area between 150° arc width sheets installed using the wet installation method and the precured installation method. Figure 4.15(a) shows two 150° arc width GFRP sheets with an average effective contact area with the concrete equal to 520 cm². Figure 4.15(b) shows the contact surface between the GFRP sheets and the concrete using the precured installation method based on Sikadur 330. The total approximate area contacted with concrete was 1312 cm² out of 1440 cm², which is 91% of the total area.

The results suggest that the precured installation method had more GFRP concrete contacting surface area over the wet installation method. More concrete surface was covered by the precured installation method because of the high viscosity of Sikadur 330 over Sikadur 300, which allowed it to stick to the concrete surface and not to slip. However, the uncovered areas helped the concrete relieve some of the applied load by having micro cracks in the concrete. The precured sheets covered most of the concrete surface area, which led to diverting the crack to pass through a nonstrengthened area. Therefore, the only factor affecting the shear capacity is the concrete property.
a) Two GFRP sheets with 150° arc width installed using wet installation method

b) One GFRP sheets with 150° arc width installed using precured installation method

Figure 4.15 Comparison between the wet installation method and the precured installation method bonding after manually peeling off the sheets

4.3.2 Ultimate load

Table 4.7 shows the ultimate loads, the percentage of shear capacity improvement compared to the control specimen and the mode of failure for four S3-Series specimens. First, S3-C had a shear capacity of 262 kN. After strengthening a S3-Series specimen with two layers of GFRP sheets with a 150° arc width, the shear capacity of specimen S3-2-450-150-W GFRP was enhanced by 51% compared to the control specimen after reaching an ultimate shear capacity of 396.5 kN. Specimen S3-2-450-150-P GFRP had an ultimate load of 405 kN, which is a 54% shear enhancement over the control specimen. Specimen S2-2-450-120-W GFRP showed an enhancement of 49% compared to the control specimen when the ultimate load reached 387 kN. The specimen strengthened with the precured installation method, S3-2-450-120-P GFRP failed at a load of 293 kN with a shear enhancement of 11% over the control specimen. The cracks in the top and bottom flanges divided the slab into two partitions instead of working as
one unit, resulting in an early shear failure in three out of five webs. The results suggest that shear enhancement depends mainly on the concrete tensile strength.

Table 4.7 Test results of four S3-Series slab specimens

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Failure Load (kN)</th>
<th>Percentage of Improvement (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-C</td>
<td>262</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S3-2-450-150-W GFRP</td>
<td>396.5</td>
<td>51</td>
<td>Brittle</td>
</tr>
<tr>
<td>S3-2-450-120-W GFRP</td>
<td>387</td>
<td>49</td>
<td></td>
</tr>
<tr>
<td>S3-2-450-150-P GFRP</td>
<td>405</td>
<td>54</td>
<td></td>
</tr>
<tr>
<td>S3-2-450-120-P GFRP</td>
<td>293</td>
<td>11</td>
<td></td>
</tr>
</tbody>
</table>

4.3.3 Load deflection behaviour

Figure 4.16 and Figure 4.17 show the load deflection difference between the wet installation method and the precured installation methods in the S3-Series measured by LVDT1 near the loading point. The control specimen, S3-C, had a linear trend that was terminated by a shear failure at 262 kN load and 2.7 mm deflection. The load deflection curves of the two specimens were strengthened with a 150° arc width using both the wet and precured installation methods are shown in Figure 4.16. Both specimens, S3-2-450-150-W GFRP and S3-2-450-150-P GFRP, had a linear behaviour in the elastic range. The stiffness in specimen S3-2-450-150-W GFRP started to decrease when a flexural crack appeared until the load stopped increasing at failure. In comparison, specimen S3-2-450-150-P GFRP had a sudden drop when one of the flanges cracked parallel to the longitudinal axis of the slab. Then, specimen S3-2-450-150-P GFRP stiffness decreased after the second crack appeared in another flange. Specimen S3-2-450-150-P GFRP continued carrying the applied load with a lower stiffness until it failed in shear.

Figure 4.17 shows the load deflection curves of the two specimens strengthened using GFRP sheets for an arc width equal to 120° using the wet and precured installation methods.
Both specimens, S3-2-450-120-W GFRP and S3-2-450-120-P GFRP, had the same linear trend in the elastic phase. However, specimen S3-2-450-120-W GFRP had five times the shear enhancement of the S3-2-450-120-P GFRP specimen and better ductility performance.

![Figure 4.16 Effect of installation method on S3-Series specimen strengthened with 150° arc width GFRP sheets](image)
Figure 4.17 Effect of installation method on S3-Series specimen strengthened with 120° arc width GFRP sheets

Figure 4.18 shows the slippage of the strands from the face of the slab web at the loading end measured by LVDT 2 for S3-Series specimens. All specimens’ strands experienced slippage immediately after the ultimate load was reached. S3-C had a maximum slippage of 0.73 mm. Specimens S3-2-450-150-W GFRP and S3-2-450-150-P had a slippage of 4.55 mm and 4.3 mm, respectively, from the face of the slab at the loaded end. For specimens S3-2-450-120-W GFRP and S3-2-450-120-P GFRP, the maximum slippage reached was 5 mm and 0.72 mm from the face of the slab at the loaded end, respectively. Specimen S3-2-450-120-P GFRP had the same strands’ slippage as the control specimen. The phenomenon suggests that the installation method does not affect the strands’ slippage and that it is connected to the deflection value.
4.3.4 Strain behaviour

Figure 4.19 shows a comparison between the load and the longitudinal compressive strain at the top part of the concrete near the loading point measured by pi gauge 4. All S3-Series specimens had the same linear trend with a slight difference in stiffness. However, each specimen reached a maximum compressive strain different than the others in the range of -159 to -255 με. Specimens S3-C, S3-2-450-150-W GFRP, S3-2-450-150-W GFRP, S3-2-450-120-W GFRP and S3-2-450-120-W GFRP had a maximum compressive strain of -253 με, -220 με, -169 με, -255 με and -159 με, respectively. All the recorded compressive strain are much smaller than the concrete crushing strain, -3500 με (CSA A23.3-14, 2014), which indicates that changing the installation method did not change the failure mode from typical shear failure.
4.4 Effect of type of FRP strengthening sheets

The test results of specimen S2-2-450-150-W GFRP is compared to the results of an identical specimen, S2-2-450, which can be identified as S2-2-450-150-W CFRP in Phase III. S2-2-450 was a S2-Series specimen tested during Phase II of the current research program (Meng, 2016). Specimen S2-2-450 is strengthened with two layers of CFRP sheets for 450 mm length and 150° arc width using the immediate wet installation method. In addition, the results of specimens S2-2-450 and S2-2-450-150-W GFRP were compared to the control specimen, S2-C. All specimens’ parameters are summarized in Table 4.8.
Table 4.8 S2-Series specimens’ parameters

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Arc Width (°)</th>
<th>Prestressing Stress (MPa)</th>
<th>Installation Method</th>
<th>Type of FRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-C</td>
<td>-</td>
<td>11.13</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S2-2-450</td>
<td>150</td>
<td>11.13</td>
<td>Wet</td>
<td>CFRP</td>
</tr>
<tr>
<td>S2-2-450-150-W GFRP</td>
<td>150</td>
<td>11.13</td>
<td>Wet</td>
<td>GFRP</td>
</tr>
</tbody>
</table>

Note. All specimens were strengthened by GFRP sheets, which were installed using the wet installation method except for the control specimen. All specimens were medium prestressing level slabs (11.13 MPa).

4.4.1 Test observations

As previously mentioned, specimens S2-C and S2-2-450-150-W GFRP failed due to a web shear tension failure at a load of 281 kN and 425 kN, respectively. Specimen S2-2-450 also experienced a web shear tension failure caused by a crack that formed at a distance of 224 mm away from the surface of the slab and an angle of 44° with respect to the slab’s longitudinal axis when the load equalled 387 kN. All crack patterns are shown in Figure 4.20.

Figure 4.20 Crack profile of S2-Series (units: mm)
4.4.2 Ultimate load

The test results of the three S2-Series specimens are summarized in Table 4.9. Both strengthened specimens reached their ultimate load when a sudden brittle tension shear failure occurred. Specimens S2-C, S2-2-450, and S2-2-450-150-W GFRP failed at 281 kN, 387 kN and 425 kN, respectively. The results suggest that strengthening PHC slab webs with two layers of CFRP for a length of 450 mm can enhance the shear capacity up to 38%, while PHC slab strengthened with GFRP sheets of the same length can enhance the shear capacity up to 52% over the control specimen.

Table 4.9 Test results of three S2-Series slab specimens

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Failure Load (kN)</th>
<th>Percentage of Improvement (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-C</td>
<td>281</td>
<td>-</td>
<td>Brittle</td>
</tr>
<tr>
<td>S2-2-450-150-W CFRP</td>
<td>389</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-150-W GFRP</td>
<td>427</td>
<td>52</td>
<td></td>
</tr>
</tbody>
</table>

4.4.3 Load deflection behaviour

Figure 4.21 shows the load deflection relation of a point near the loading position for three S2-Series specimens measured by LVDT 1. Initially, both specimens had the same elastic trend. However, after the elastic phase, each specimen had its unique plastic behaviour, as the displacement and the ultimate loads varied from one specimen to another. As previously mentioned, specimen S2-C failed at the 280.5 kN load and had a maximum vertical displacement of 2.8 mm. However, specimen S2-2-450 slope shows a linear elastic behaviour until the first crack appeared at a load of 280 kN. After the first crack, the CFRP sheets started to carry the shear stress. The stiffness of the specimen kept decreasing until failure with an ultimate load of 387 kN and a maximum vertical displacement of 5 mm. Finally, specimen S2-2-450-150-W GFRP had a linear behaviour until a flexural crack was formed, which appeared at a load of 290
kN. The slope of the load deflection curve of specimen S2-2-450-150-W GFRP shows a decrease in the stiffness. Simultaneously, the GFRP sheets of this specimen started to carry the shear stress up to an ultimate load of 425 kN and a deflection of 26.7 mm.

This set of results suggests that CFRP and GFRP sheets did not help in postponing the formation of the first crack. However, both FRP sheets helped in increasing the maximum deflection up to ten times compared to the maximum deflection of the control specimen. Additionally, strengthening PHC slab webs with GFRP sheets increases the loading carrying capacity and allows for a better ductility compared to PHC slabs strengthened with CFRP sheets.

Figure 4.21 Load versus deflection near loading point for S2-Series specimens
Figure 4.22 shows the strand slippage from the surface of the concrete at the loading end, recorded by LVDT 2. Both S2-C and S2-2-450 specimens had a gradual increase in the slippage with the increasing load with maximum slippages of 0.76 mm and 1.7 mm, respectively. The stands of specimen S2-2-450-150-W GFRP had a unique slip behaviour. The strands started to slip after the first flexural crack was formed and continued to a maximum value of 5.9 mm at the failure.

![Graph showing load-end slip behaviour for S2-Series specimens](image)

**Figure 4.22 Load end slip behaviour for S2-Series specimens**

### 4.5 Cost Analysis

In construction engineering, the design requirements of a structure are determined based on two factors: the loading capacity and the cost of the project. Since FRP composite is a new technology, the design codes are conservative when it comes to design and, it is more expensive than traditional reinforcement approaches. Therefore, a cost analysis of the new shear
strengthening technique for the PHC slabs is required to give a realistic estimation. The cost estimation will include a detailed evaluation of the required material.

In general, the nature of the application includes an adhesive material that can affect the skin and is hard to get off surfaces and clothes. Therefore, every time an installation takes place, new production and protection tools are required. All disposable equipment and their costs in Canadian dollars (CAD) are summarized in Table 4.10.

Table 4.10 General items required for the strengthening process

<table>
<thead>
<tr>
<th>Item</th>
<th>Price (CAD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brush</td>
<td>1.5</td>
</tr>
<tr>
<td>Light Weight Disposal Gown (20 per pack)</td>
<td>10</td>
</tr>
<tr>
<td>High Duty Gloves (per pair)</td>
<td>2</td>
</tr>
<tr>
<td>Masks (50 per pack)</td>
<td>6</td>
</tr>
</tbody>
</table>

4.5.1 Wet Installation method

The wet installation method requires only one type of epoxy, Sikadur 300, and the FRP sheets. Sikadur 300 comes in two parts with a total size of 15 litres with a cost around 530 CAD. Sikadur 300 is used to saturate the core surfaces, which usually consumes around 0.4 litres for the four cores. Then, Sikadur 300 is used to saturate the FRP sheets, which usually consumes up to 1.63 litres of the epoxy based on the area of the sheets. The GFRP sheets come in a roll of 127 cm by 45.7 m or 9.1 m. The 127 cm by 9.1 m roll costs 384 CAD. The sheet area varies based on the arc length, which is in the range of 260 mm to 381 mm. A more detailed accounting of the materials involved and their costs are listed in Table 4.11.
Table 4.11 Wet installation method quantities of GFRP sheets take off per specimen

<table>
<thead>
<tr>
<th>Arc Width (degrees) – Arc Length (mm)</th>
<th>Sikadur 300 (litres)</th>
<th>Sikadur 300 Price (CAD)</th>
<th>GFRP Sheets Price (CAD)</th>
<th>Total (CAD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>180 – 381</td>
<td>2</td>
<td>71</td>
<td>129</td>
<td>200</td>
</tr>
<tr>
<td>150 – 320</td>
<td>1.8</td>
<td>64</td>
<td>108</td>
<td>172</td>
</tr>
<tr>
<td>120 – 260</td>
<td>1.6</td>
<td>57</td>
<td>88</td>
<td>145</td>
</tr>
</tbody>
</table>

Note. All sheets are 450 mm in length. The amounts stated are for sixteen GFRP sheets.

Just like GFRP sheets, Sika Wrap-900C is the CFRP sheets used in Phase II of the current research by Meng (2016). CFRP sheets are known to be stronger than GFRP sheets. However, the CFRP sheets used cost 70 CAD/m², which is more expensive than the GFRP sheets. Additionally, there is no difference when it comes to saturating the sheets with Sikadur 300. Table 4.12 summarizes all the material required and their costs to strengthen one side of a PHC slab.

Table 4.12 Wet installation method quantities of CFRP sheets take off per specimen

<table>
<thead>
<tr>
<th>Arc Width (degrees) – Arc Length (mm)</th>
<th>Sikadur 300 (litres)</th>
<th>Sikadur 300 Price (CAD)</th>
<th>CFRP Sheets Price (CAD)</th>
<th>Total (CAD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>180 – 381</td>
<td>1.9</td>
<td>68</td>
<td>189</td>
<td>257</td>
</tr>
<tr>
<td>150 – 320</td>
<td>1.7</td>
<td>61</td>
<td>161</td>
<td>222</td>
</tr>
<tr>
<td>120 – 260</td>
<td>1.5</td>
<td>54</td>
<td>133</td>
<td>187</td>
</tr>
</tbody>
</table>

Note. All sheets are 450 mm in length. The amounts used are for sixteen GFRP sheets.

4.5.2 Precured installation method

The precured installation method requires two types of epoxy that are used separately. The precured installation method also requires the sheets to dry and take the circular shape of the web. For the 305 mm depth slabs, the sheets can be shaped using the inner shape of 10 in sonotube. After 24 hours from saturating the GFRP sheets, Sikadur 330, which is a highly viscous epoxy, is used to establish contact between the concrete and GFRP surfaces and between the next FRP layer. To establish a contact surface between the concrete and the GFRP sheets, an
average of 0.5 kg Sikadur 330 is used. The second layer consumes around 0.3 kg Sikadur 330. Sikadur 330 is a two parts epoxy, which weighs 5 kg per package and costs 150 CAD. Detailed quantities take out and estimated prices are all summarized in Table 3.8.

Table 4.13 Precured installation method quantities of GFRP sheets take out per specimen

<table>
<thead>
<tr>
<th>Arc Width (°) – Arc Length (mm)</th>
<th>Sikadur 300 (litres)</th>
<th>Sikadur 300 Price (CAD)</th>
<th>Sikadur 330 (kg)</th>
<th>Sikadur 330 Price (CAD)</th>
<th>GFRP Sheets (m²)</th>
<th>GFRP Sheets Price (CAD)</th>
<th>Total (CAD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 – 320</td>
<td>1.3</td>
<td>46</td>
<td>7.5</td>
<td>225</td>
<td>2.3</td>
<td>108</td>
<td>379</td>
</tr>
<tr>
<td>120 – 260</td>
<td>1</td>
<td>36</td>
<td>7</td>
<td>210</td>
<td>1.872</td>
<td>88</td>
<td>334</td>
</tr>
</tbody>
</table>

Note. All sheets are 450 mm in length. The amounts used are for sixteen GFRP sheets.

CFRP sheets can be installed by the precured installation method as well. The same amount of Sikadur 300 and Sikadur 330 required when installing the GFRP sheets is used to install the CFRP sheets. However, the only difference in the cost will be in the CFRP sheet price. Table 4.13 summarizes the cost of installing CFRP sheets using the precured method.

Table 4.14 Precured installation method quantities of CFRP sheets take out per specimen

<table>
<thead>
<tr>
<th>Arc Width (°) – Arc Length (mm)</th>
<th>Sikadur 300 (litres)</th>
<th>Sikadur 300 Price (CAD)</th>
<th>Sikadur 330 (kg)</th>
<th>Sikadur 330 Price (CAD)</th>
<th>CFRP Sheets (m²)</th>
<th>CFRP Sheets Price (CAD)</th>
<th>Total (CAD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 – 320</td>
<td>1.2</td>
<td>44</td>
<td>7.5</td>
<td>225</td>
<td>2.3</td>
<td>161</td>
<td>430</td>
</tr>
<tr>
<td>120 – 260</td>
<td>0.9</td>
<td>33</td>
<td>7</td>
<td>210</td>
<td>1.872</td>
<td>133</td>
<td>376</td>
</tr>
</tbody>
</table>

Note. All sheets are 450 mm in length. The amounts used are for sixteen GFRP sheets.

4.5.3 Summary

In summary, choosing an installation method and strengthening material size depends on the design requirements and cost. Therefore, proceeding with a specific design will include tools, material and labor costs. In addition, each installation method has its own procedure. The approximate total cost of installing the GFRP or CFRP sheets using the wet installation method
can vary between 145 CAD and 257 CAD. The precured installation method can cost between 334 CAD and 430 CAD for both CFRP and GFRP sheets. The above estimation shows that the cost of the wet installation method is almost half of that of the precured installation method. The difference in cost is mainly because of the usage of two types of epoxies in the precured installation method instead of only one compared to the wet installation method. Finally, the CFRP sheets are more expensive than the GFRP sheets.
CHAPTER 5: CODE ANALYSIS

Code predictions are used to design structural members that are effective and economical to carry the required loads. In this chapter, the shear capacities of the tested 11 specimens will be predicted based on the codes of the American Concrete Institute (ACI) and Canadian Standards Association (CSA). In addition, code predictions will be compared with the experimental results.

5.1 ACI Code

The ACI code provides design provisions and guidelines for the calculation of the shear capacity of concrete members strengthened with externally bonded FRP sheets. The ACI code calculates the shear contribution of concrete and FRP sheets separately. The concrete and the GFRP sheets’ contribution to the shear capacity are calculated based on the ACI-318-14 and ACI-440.2R-15, respectively, then both values are added to find the predicted shear capacity of the strengthened PHC slabs.

5.1.1 ACI 318-14 and ACI 440.2R-15

ACI-318-14 was used to calculate the shear capacity of the three tested control specimens (S1-C, S2-C and S3-C) in both Phase II and Phase III. Specimens S1-C, S2-C and S3-C were predicted to have an ultimate load of 226 kN, 262 kN and 284 kN, respectively. Then, the shear capacity contributed by the GFRP sheets was calculated separately using ACI.440-2R-15. In Phase III, the GFRP sheets were 450 mm long, two layered and either 120°, 150° or 180° in arc widths. The code predictions showed that the sheets should contribute to the shear capacity of the strengthened PHC slabs in the range between 53.2 kN and 61.6 kN. The predicted shear capacity of the strengthened specimens was estimated to be in the range between 287.6 kN and 346 kN. The experimental results varied between 262 kN and 443 kN. Table 5.1 summarizes the shear
contribution and the code prediction of each studied specimen and the experimental results. Detailed calculations are given in the appendices.

Table 5.1 ACI 318-15 and ACI 440.2r-15 shear prediction results and the experimental results for all specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>ACI code</th>
<th>Experimental (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( V_c ) (kN) (by ACI 318-15)</td>
<td>( V_{frp} ) (kN) (by ACI 440.2r-15)</td>
</tr>
<tr>
<td>S1-C</td>
<td>226</td>
<td>-</td>
</tr>
<tr>
<td>S1-2-450-180 GFRP</td>
<td>226</td>
<td>61.6</td>
</tr>
<tr>
<td>S2-C</td>
<td>262</td>
<td>-</td>
</tr>
<tr>
<td>S2-2-450-120 GFRP</td>
<td>262</td>
<td>53.2</td>
</tr>
<tr>
<td>S2-2-450-150 GFRP</td>
<td>262</td>
<td>59.4</td>
</tr>
<tr>
<td>S2-2-450-180 GFRP</td>
<td>262</td>
<td>61.6</td>
</tr>
<tr>
<td>S3-C</td>
<td>284</td>
<td>-</td>
</tr>
<tr>
<td>S3-2-450-120 GFRP</td>
<td>284</td>
<td>53.2</td>
</tr>
<tr>
<td>S3-2-450-150 GFRP</td>
<td>284</td>
<td>59.4</td>
</tr>
<tr>
<td>S3-2-450-180 GFRP</td>
<td>284</td>
<td>61.6</td>
</tr>
</tbody>
</table>

Note. * Duplicated specimen.

5.1.2 Total shear capacity by the ACI code versus. experimental results

Figure 5.1 shows the ratio of the theoretical to experimental results based on the ACI code for PHC slabs strengthened with GFRP sheets. The predicted shear capacities of the strengthened PHC slabs values show an underestimation of the overall shear capacity compared to the experimental results for most of the specimens except for two strengthened PHC slabs, which had a reasonable estimate. The difference between the two varies between -29% and +15%.
5.2 CSA Code

The CSA organization is responsible for publishing standards similar to the ACI code. The CSA codes can be used to determine the shear capacity of a strengthened PHC slab with GFRP sheets via CSA-A23.3-14 and CSA-S806-12. The CSA-A23.3-14 determines the shear capacity of concrete only, while CSA-S806-12 determines the shear capacity contributed by the GFRP sheets.

5.2.1 CSA-A23.3-14 and S806-12

The shear capacities of the three control specimens were calculated but with the CSA-A23.3-14 code. The code predicted shear capacity of S1-Series specimen to be 141 kN and both S2-Series and S3-Series specimens to be 163 kN. The results show that the CSA-A23.3-14
underestimates the shear capacities of PHC slabs compared to the experimental results. In addition, the contribution of the GFRP sheets in carrying the shear capacity was calculated using CSA-S806-12. The code predicts the GFRP sheets’ contribution to the shear capacity with different arc widths is 98 kN. The contribution is the same for all arc widths because the depth counted in the calculations is the effective shear depth of steel. Therefore, the total estimated shear capacity is equal to 261 kN for all the strengthened specimens except for specimen S1-2-450-180 W GFRP, which was estimated to be 239 kN. The experimental results varied between 262 kN and 443 kN. Table 5.2 summarizes the shear capacities calculated by CSA-A23.3-14 and CSA-S806-12 for all the tested specimens. Detailed calculations are given in the appendices.

Table 5.2 CSA-A23.3-14 and CSA-S806-12 shear prediction and the experimental results for all specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>CSA Code</th>
<th>Experimental (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_e$ (kN) (by CSA-A23.3-14)</td>
<td>$V_{frp}$ (kN) (by CSA-S806-12)</td>
</tr>
<tr>
<td>S1-C</td>
<td>141</td>
<td>-</td>
</tr>
<tr>
<td>S1-2-450-180 GFRP</td>
<td>141</td>
<td>98</td>
</tr>
<tr>
<td>S2-C</td>
<td>163</td>
<td>-</td>
</tr>
<tr>
<td>S2-2-450-120 GFRP</td>
<td>163</td>
<td>98</td>
</tr>
<tr>
<td>S2-2-450-150 GFRP</td>
<td>163</td>
<td>98</td>
</tr>
<tr>
<td>S2-2-450-180 GFRP</td>
<td>163</td>
<td>98</td>
</tr>
<tr>
<td>S3-C</td>
<td>163</td>
<td>-</td>
</tr>
<tr>
<td>S3-2-450-120 GFRP</td>
<td>163</td>
<td>98</td>
</tr>
<tr>
<td>S3-2-450-150 GFRP</td>
<td>163</td>
<td>98</td>
</tr>
<tr>
<td>S3-2-450-180 GFRP</td>
<td>163</td>
<td>98</td>
</tr>
</tbody>
</table>

Note. * Duplicated specimen

5.2.2 Total shear capacity by the CSA code versus experimental results

Figure 5.2 shows a comparison between the total capacity predicted by the CSA code and the experimental results of each specimen. The graph shows a large difference between the code predicted values and the experimental results. The total shear capacity predicted by the CSA
code is lower than all the experimental results for most of the strengthened specimens. The difference between the predicted value and the actual shear capacity varied between 0.6 and 0.9 times. This considerable difference is due largely to the underestimation of the CSA code on the shear capacity contributed by the PHC slabs.

![Figure 5.2 Ratio of the theoretical to experimental results based on CSA code for PHC slabs strengthened with GFRP sheets](image-url)
CHAPTER 6: CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

Shear strengthening of PHC slabs by installing GFRP sheets on all the inner surfaces of the slab cores has been studied. The total number of specimens used in the experimental work was 11 full width PHC slab specimens. Ten out of the 11 specimens were strengthened using GFRP sheets to investigate the feasibility and effectiveness of using GFRP sheets on the shear capacity. The tested parameters included the prestressing level of the slab, the arc width of the strengthened area and the installation method. Strengthening PHC slabs with GFRP sheets in shear can be improved with up to 58% compared to the control specimen.

The main conclusions gained from both the experimental work and code analysis are summarized as follows:

1. The effect of PHC slab prestressing levels on the shear capacity of the strengthened specimen has been studied. The three levels of prestressed slabs are: the low-prestressing level (6 prestressing strands), the medium prestressing level (8 prestressing strands), and the high-prestressing level (8 prestressing strands in tension zone and 2 prestressing strands in compression zone). Results show that the prestressing forces did not affect the shear capacity of the strengthened PHC slabs.

2. The effect of strengthening width was evaluated on three arc widths with an arc angle of $120^\circ$, $150^\circ$ and $180^\circ$. The results showed that strengthening PHC slabs with a full arc width ($180^\circ$) GFRP sheets or reducing the GFRP sheets arc width to $120^\circ$ do not have a sizeable impact on the shear capacity as long as the shear critical zone is covered.

3. The specimens strengthened using the wet installation method showed shear enhancement between 13% and 58%. The precured method showed shear enhancement between 11% and
52%. However, using different installation methods did not affect the contribution of the GFRP sheets to the shear capacity of the strengthened PHC slabs.

4. PHC slabs strengthened using GFRP sheets showed competitive results to the PHC slabs strengthened using CFRP sheets. PHC slabs strengthened with two layers of 450 mm long CFRP sheets showed a shear enhancement of 31% to 38%. In addition, PHC slabs strengthened with two layers of 450 mm long GFRP sheets had a shear enhancement in the range of 36% to 58%.

5. The shear capacity of the tested specimens was predicted using ACI 318-14 and ACI 440.2r-15. Both the ACI 318-14 and ACI 440.2r-15 underestimated the shear capacity of most strengthened PHC slab specimens.

6. The CSA-A23.3-14 and CSA-S-806-12 were also used to calculate the shear capacities of the tested specimens. The CSA-A23.3-14 showed an underestimation of the concrete contribution to the shear capacity. The CSA-S-806-12 showed a fair estimation while predicting the FRP shear contribution. The shear capacity predicted by both codes underestimated the total shear capacity of the strengthened slabs.

6.2 Recommendations for Future Work

The following recommendations are based on the results and conclusions achieved in the current and previous studies:

1. Additional parameters need to be evaluated in the next phases, For example: other prestressing levels, noncircular core PHC slabs and other types of FRP sheets and epoxy.

2. In most of the cases, only one experimental test was conducted for each studied parameter. More experimental tests are needed to confirm the parametric effect found in the current study.
APPENDIX A  Calculation of shear capacity of prestressed hollow core slabs using ACI 318-14

Prestressing Steel diameter = 13 mm (0.511 in); $f_{pu} = 1860$ MPa = 270 ksi; initial Stress = 70% $f_{pu}$; Prestressing losses = 15%; $f'_c$ = 55 MPa (7.25 ksi); $L = 177.2$ in; $b_w = 205$ mm = 8.07 in; $d = 274$ mm = 10.8 in; $V_p = 0$; Area of slab = 1.79($10^5$) mm$^2$.

1. Formulate the nominal shear strength provided by the concrete equation:

$$V_{cw} = (3.5 \sqrt{f'_c} + 0.3 f_{pc}) b_w d_p + V_p$$

$$V_{cw} = (3.5 \sqrt{7251(\text{psi})} + 0.3 f_{pc}) (5)(1.614(\text{in}))(10.8(\text{in})) + 0$$

$$= 25975.5 + 26.14 f_{pc}$$

$f_{pc}$ is calculated as a function of the transfer of prestress force into the section along the span.

2. Calculate the prestress forces transfer length:

Transfer length = $50d_p = 50(0.5(\text{in})) = 25$ in

Bearing length = 3.35 in

Full prestress transfer is achieved when $(25(\text{in}) - 3.35(\text{in})) = 21.65$ in from the face of support.

3. Calculate the effective forces in the prestressing strands:

$$A_{ps}f_{se} = (6)(0.205)(270000)(0.7)(1 - 0.15) \left( \frac{x + 3.35}{25} \right)$$

At a shear span to depth ratio equal to 2.5; $x = 762.5$ mm = 30 in:
\[ A_{ps}f_{se} = (6)(0.205(in^2))(270000(Psi))(0.7)(1 - 0.15)(\frac{30(in) + 3.35(in)}{25(in)}) \]

\[ = 263597.734 \text{ lb} \]

4. The compressive stress in concrete at centre point:

\[ A_{slab} = 1.79 \times 10^5 \text{ mm}^2 = 277.45 \text{ in}^2 \]

\[ f_{pc} = \frac{A_{ps}f_{se}}{A_{slab}} = \frac{263597.73}{277.45} = 950 \]

5. Calculate the nominal shear strength provided by concrete:

\[ V_{cw} = 25975.5 + 26.14 f_{pc} = 25975.5 + 26.14(950) = 50808 \text{ lb (226 kN)} \]

<table>
<thead>
<tr>
<th>Specimen Series</th>
<th>( A_{ps}f_{se} )</th>
<th>( A_{slab} (in^2) )</th>
<th>( f_{pc} )</th>
<th>( V_{cw} (lb) )</th>
<th>( V_{cw} (kN) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-Series</td>
<td>263597.73</td>
<td>277.45</td>
<td>950</td>
<td>50808</td>
<td>226</td>
</tr>
<tr>
<td>S2-Series</td>
<td>351463.64</td>
<td>277.45</td>
<td>1266.76</td>
<td>59068</td>
<td>262</td>
</tr>
<tr>
<td>S3-Series</td>
<td>403325</td>
<td>277.45</td>
<td>1453</td>
<td>63974</td>
<td>284</td>
</tr>
</tbody>
</table>

**APPENDIX B** Calculation of shear capacity of prestressed hollow core slabs using CSA-A23.3-14

Given: L = 4.5 m; \( f_{pu} = 1860 \text{ MPa} \); \( f'_c = 55 \text{ MPa} \); \( a_g = 20 \text{ mm} \); \( E_p = 196500 \text{ MPa} \); Strands pulled to 70% \( f_{pu} \); Total losses = 15%; Strands diameter = 13 mm; Web thickness at the middle of the slab = 41 mm; Area of slab = \( 1.79(10^5) \text{ mm}^2 \).

1. Determine \( b_w \)
Average additional width of the core is estimated to be $2\% = 0.02(121.5 \text{ (mm)}) = 2.42$ mm:

$$b_w = 41(2) + 3(41 + 2(2.42)) = 219 \text{ mm}$$

2. Determine $d_v$ use the largest of these two:

$$0.9 d = (0.9)(305 - 31) = 246.6 \text{ mm}$$

$$0.72 h = (0.72)(305) = 219.6 \text{ mm}$$

3. Determine $S_{ze}$, when $Ag$ is 16 mm:

$$S_{ze} = \frac{35 S_z}{15 + a_g} = \frac{(35)(246.6 \text{ (mm)})}{15 + 16} = 278.4 \text{ mm} > 0.85 S_z$$

4. Formulate $\beta$:

$$\beta = \left[ \frac{(0.4)}{(1 + 1500 \varepsilon_x)} \right] \left[ \frac{(1300)}{(1000 + S_{ze})} \right]$$

$$= \left[ \frac{(0.4)}{(1 + 1500 \varepsilon_x)} \right] \left[ \frac{(1300)}{(1000 + 278.4)} \right] = \frac{0.406}{(1 + 1500 \varepsilon_x)}$$

5. A concentrated load based on the experimental work shall be 290 kN at 762.5 mm from the face of the slab:

a. Determine the reaction near the loading point:

$$R_1 = \frac{(290 \text{ (kN)})(3.7375 \text{ (m)})}{4.5 \text{ (m)}} = 240.9 \text{ kN}$$

$$\therefore V_f = 240.9 \text{ kN @ } x = 335 \text{ mm}$$

$$& M_f = (0.335 \text{ (m)})(240 \text{ (kN)}) \approx 80.4 \text{ kN.m}$$

6. Calculate $\varepsilon_x$:

$$\varepsilon_x = \frac{M_f}{d_v} + V_f - V_p + 0.5N_f - A_p f_{po}$$

$$= \frac{M_f}{d_v} + V_f - A_p f_{po}$$

$$= \frac{M_f}{2A_p E_p + 2A_{ct} E_c}$$
\[
\frac{80.4 \times 10^6 (N \cdot mm)}{246.6 (mm) + 240(1000)N - (6)(132.7(mm^2))(0.7)(335 \times 650)(1860(MPa))} \\
= \frac{0.0001015 \geq 0}{2(196500(MPa))(6)(132.7(mm^2))}
\]

7. Determine \( \beta \):

\[
\beta = \frac{0.406}{(1 + 1500\varepsilon_x)} = \frac{0.406}{(1 + (1500(0.0001015)))} = 0.352
\]

8. Determine \( V_c \):

\[
V_c = \lambda \beta \sqrt{f'_c b_w d_v} = 0.352 \sqrt{55(MPa)}(246.6(mm))(219(mm)) = 141 kN
\]

Table B.1 Summary of prestressed hollow core slabs calculations using CSA-A23.3-14

<table>
<thead>
<tr>
<th>Specimen Series</th>
<th>( S_{ze} ) (mm)</th>
<th>( \varepsilon_x )</th>
<th>( \beta )</th>
<th>( V_c ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-Series</td>
<td>278.4</td>
<td>0.0001015</td>
<td>0.352</td>
<td>141</td>
</tr>
<tr>
<td>S2-Series</td>
<td>278.4</td>
<td>0</td>
<td>0.406</td>
<td>162.6</td>
</tr>
<tr>
<td>S3-Series</td>
<td>278.4</td>
<td>0</td>
<td>0.406</td>
<td>162.6</td>
</tr>
</tbody>
</table>

**APPENDIX C** Calculation of shear capacity of the GFRP sheets using S806-12

Ply thickness of GFRP (\( t_{FRP} \)) = 1.016 mm; GFRP Tensile modulus (\( E_{FRP} \)) = 26119 MPa; \( f'_c \), GFRP tensile elongation (\( \varepsilon_F \)) = 0.0006; \( w_f \) = 450 mm; Crack angle (\( \theta \)) = 35°.

1. Determine \( d_v \) to be the larger of:

\[
0.9h = 0.9(290) = 261 mm
\]

And \( 0.72h = 0.72(305) = 219.6 mm \)

\[\therefore d_v = 261 mm\]

2. Determine the area of FRP (\( A_{fv} \)):
\[ A_{fv} = 2ntfwf = 2(2)(1.016(mm))(450(mm)) = 1828.8 \, mm^2 \]

3. Determine the shear contribution of the GFRP sheets \((V_f)\):

\[
V_f = \frac{A_F E_F \varepsilon_f d_v (\cot \theta + \cot \alpha_f) \sin \alpha_f}{S_F}
\]

\[
= \left( \frac{1828.8 \, (mm^2)}{(26119 \, (MPa))((0.0006)(261(mm)))(\cot 35^\circ + \cot 9^\circ) \sin 90^\circ}{450(mm)} \right)
\]

\[ = 24507 \, N \]

\[ V_{f\text{Total}} = 4(24507) = 98 \, kN \]

Table C.1 Summary of prestressed hollow core slabs calculations using CSA-S806-12

<table>
<thead>
<tr>
<th>Arc Angle (°)</th>
<th>(d_v) (mm)</th>
<th>(A_{fv}) (mm²)</th>
<th>(V_f) (kN)</th>
<th>(V_{f\text{Total}}) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>261</td>
<td>1828.8</td>
<td>24.5</td>
<td>98</td>
</tr>
<tr>
<td>150</td>
<td>261</td>
<td>1828.8</td>
<td>24.5</td>
<td>98</td>
</tr>
<tr>
<td>180</td>
<td>261</td>
<td>1828.8</td>
<td>24.5</td>
<td>98</td>
</tr>
</tbody>
</table>

APPENDIX D  Calculation of shear capacity of GFRP sheets using ACI-440.2r-15

Ply thickness of GFRP \((t_{FRP}) = 1.016\, mm\); GFRP Tensile modulus \((E_{FRP}) = 26119 \, MPa\); \(f'_c = 55 \, MPa\); GFRP tensile elongation \((\varepsilon_{fu}) = 0.0006\); \(wf = 450 \, mm\).

1. Determine the effective depth:

\[ d_{fv} = 2(121) \sin(150) = 233.75 \, mm \]

2. Determine the area of FRP \((A_{fv})\):

\[ A_{fv} = 2ntfwf = 2(2)(1.016(mm))(450(mm)) = 1828 \, mm^2 \]
3. Determine the effective stress in FRP ($f_{fe}$):

$$f_{fe} = (0.0006)(26119 \text{ (MPa)}) = 14.8 \text{ N/mm}^2$$

4. Determine the shear contribution of the GFRP sheets:

$$V_f = \frac{2nt_w E_f \varepsilon_{fe}(\cos \alpha_f + \sin \alpha_f)d_f}{S_f}$$

$$V_f = \frac{(1828(\text{mm}^2))(14.8(\frac{N}{\text{mm}^2}))(1)(233(\text{mm}))}{450(\text{mm})} = 14839 \text{ N}$$

$$V_{f\text{Total}} = (4)(14839(\text{N})) = 59.4 \text{ kN}$$

<table>
<thead>
<tr>
<th>Arc Angle (°)</th>
<th>$d_v$ (mm)</th>
<th>$A_{fv}$ (mm$^2$)</th>
<th>$V_f$ (kN)</th>
<th>$V_{f\text{Total}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>209</td>
<td>1828.8</td>
<td>13.3</td>
<td>53.2</td>
</tr>
<tr>
<td>150</td>
<td>233</td>
<td>1828.8</td>
<td>14.8</td>
<td>59.4</td>
</tr>
<tr>
<td>180</td>
<td>242</td>
<td>1828.8</td>
<td>15.4</td>
<td>61.6</td>
</tr>
</tbody>
</table>
APPENDIX E  Wet Installation method in detail

a) Cut the sheets  
b) Mix Sikadur 300  
c) Saturate GFRP sheets with Sikadur 300

d) Saturate concrete surface with Sikadur 300  
e) Place wet GFRP sheet on the web  
f) Complete first layer for all webs

g) Place second wet GFRP sheet  
h) Complete second layer for all webs

Figure E.1 Wet installation method.
APPENDIX F  Pre-cured installation method in detail

Figure F.1 Pre-cured installation method
APPENDIX J  GFRP sheets with different arc widths

Figure J.1 GFRP sheets with different arc width angles using wet installation method

Figure J.2 GFRP sheets with different arc width angles using precured installation method

APPENDIX H  Crack Profiles

Figure H.1 Crack profiles for S1-Series specimen, S1-2-450-180-W GFRP
Figure H.2 Crack profiles for S2-Series specimens

a) S2-2-450-180-W GFRP

b) S2-2-450-180-W GFRP 2nd

c) S2-2-450-150-W GFRP

d) S2-2-450-120-W GFRP
a) S3-C

b) S3-2-450-180-W GFRP

c) S3-2-450-150-W GFRP

d) S3-2-450-120-W GFRP
e) S3-2-450-150-P GFRP

f) S3-2-450-120-P GFRP

Figure H.3 Crack profiles S3-Series specimens
REFERENCE


VITA AUCTORIS

NAME: Amr Alaa Abdelaal

PLACE OF BIRTH: Kafr El Shikh, Egypt

YEAR OF BIRTH: 1993

EDUCATION:

American University of Sharjah, Sharjah, UAE

2011–2015 Bachelor’s degree in Civil Engineering

University of Windsor, Windsor, ON, Canada

2012–2017 M.A.Sc. Civil Engineering/Structural