Shear Strengthening of Prestressed Hollow Core Slabs Using Externally Bonded Carbon Fiber Reinforced Polymer Sheets

Xianzhe Meng

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Shear Strengthening of Prestressed Hollow Core Slabs Using Externally Bonded Carbon Fiber Reinforced Polymer Sheets

By

Xianzhe Meng

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

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Shear Strengthening of Prestressed Hollow Core Slabs Using Externally Bonded Carbon Fiber Reinforced Polymer Sheets

By

Xianzhe Meng

APPROVED BY:

______________________________________________
N. Zamani, Outside Dept. Reader
Department of Mechanical, Automotive & Materials Engineering

______________________________________________
F. Ghrib Dept. Reader
Department of Civil and Environmental Engineering

______________________________________________
S. Cheng, Principal Advisor
Department of Civil and Environmental Engineering

______________________________________________
A. El Ragaby, Co-Advisor
Department of Civil and Environmental Engineering

June 14, 2016
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ABSTRACT

Precast prestressed hollow core slabs are widely used for concrete structures all over the world. Since no shear stirrup arranged in PHC slabs, the shear resistance is only provided by concrete itself. In some cases, the slab could possibly be subjected to concentrated or line load. Thus, shear failure is likely to occur at the region close to supports. The objectives of this study include explore a new shear strengthening technique by externally bonded Carbon Fiber Reinforced Polymer composite sheets to the internal voids surface of the slabs to enhance its shear capacity. Experimental tests and numerical simulations are carried out on full-size PHC slabs to evaluate the effectiveness of this new method. The studied parameters include the prestressing level, the length and the thickness of the applied CFRP sheets. Both experimental and numerical simulation results showed considerable shear capacity and ductility enhancement by applying this shear strengthening technique.

Key words: PHC slabs, shear capacity, CFRP and finite element simulation.
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CHAPTER 1 INTRODUCTION

1.1 Background

A hollow core slab (Figure 1.1) is a precast prestressed structural member which has several voids extended through the member length to reduce its self-weight and increase economic benefit. In general, this type of slab is widely used as floor deck in office, residential buildings and parking structures because of its thermal and sound isolation characteristics. In addition, due to its light self-weight and high load bearing capacity, hollow core slabs is favorable choice for long spans and heavy live loads. During construction, hollow core slab units are placed side-by-side to each other to form a continuous floor system. The gap between the adjacent slabs is filled with non-shrink concrete. Finally, a layer of concrete surface is placed on the top of the slabs.

Figure 1.1 Prestressed hollow core slabs

The manufacturing process of precast prestressed hollow core (PHC) slabs consists
of 6 steps in total. They include preparing bed, running of strands, tensioning strand, pouring, cutting notches and inspection. The whole casting is made on a long casting bed. 

Before casting a slab, the steel strands should be tensioned first and this step is finished by wire stressing jack. Then, a casting machine extruder (Figure 1.2) is used to extrude concrete as molding while moving forward and the voids of PHC slab are formed by the tubes at the bottom of the machine. This machine is able to cast slabs of different depth from 150mm to 400mm. In order to obtain the required length of a PHC slab, a standard concrete saw is used to cut the slab.

![Figure 1.2 PHC slab extruder](http://www.xingyumachine.cn/goods-detail-id_39.html)

A PHC slab only has bottom reinforcements, which can reduce around 40% usage of steel. Furthermore, the voids in the slabs can save approximate 30% of concrete (Mahmut, 2011). In summary, easy construction technology, light self-weight, low cost and material usage make PHC slabs a very popular structural member in engineering application.
1.2 Motivations

PHC slabs have wide applications. This type of slab is generally designed for resisting bending moment under uniformly distributed load. However, in some cases, the slab could possibly be subjected to concentrated load or line load, which can cause shear failure at the region close to support (Figure 1.3). Traditional methods of improving shear capacity of PHC slabs include increasing slab thickness and filling the voids. According to ACI 318-08 (ACI Committee 318, 2014), increasing thickness of slabs beyond a certain value would have negative effect on the shear capacity. On the other hand, filling voids would affect the advantages of light self-weight and low cost. The manufacturing process of PHC slabs does not allow for arranging shear reinforcement during slab fabrication. Without shear reinforcement, PHC slabs can only rely on the tensile strength of concrete to resist shear force. So strengthening shear capacity of a PHC slab is the best way to avoid shear failure of PHC slab while still retains its merits.

![Figure 1.3 Typical shear failure of the PHC slab](image)

Currently, Fiber Reinforced Polymer (FRP) Composite is widely used to strengthen and rehabilitate many structures. It is a kind of high strength composite material made of a polymer matrix reinforced with fibers. The FRP composite has the advantages in terms
of its high tensile strength, light self-weight, flexible shape, waterproof capability, as well as good fatigue and corrosion resistance capacity. Bonding FRP composite sheets to structural members would hardly affect the original dimension and self-weight of the member, but could help to efficiently increase its load carrying capacity. Studies on using FRP composite sheets to strengthen shear capacity of PHC slabs are scarce. Most existing researches focused on using FRP composite to strengthen shear or flexural capacity of reinforced concrete beams or slabs and rehabilitate existing structures.

In some shear strengthening studies, the role of bonded FRP sheets was considered to be similar as the shear stirrups. Thus, it is worth to study the feasibility and effectiveness of applying CFRP sheets to PHC slabs for shear strengthening. This thesis will propose a new shear strengthening approach which attaches Carbon Fiber Reinforced Polymer (CFRP) composite sheets to the inner surface of each void of a PHC slab to increase its shear capacity (Figure 1.4).

Figure 1.4 Strengthening PHC slab by FRP sheets
1.3 Objectives

The objectives of this study are proposed as follows:

1. Evaluate the feasibility and effectiveness of a new shear strengthening technique for PHC slabs of which Carbon Fiber Reinforced Polymer (CFRP) composite sheets are bonded to the surface of internal voids of the slab to increase its shear capacity.

2. Investigate the influence of several parameters on the shear strengthening effect of PHC slabs through experimental test, which include the prestressing level of the PHC slabs, the thickness and the length of applied CFRP sheets.

3. Conduct a finite element simulation to understand the shear behavior of full width PHC slabs equipped with externally bonded CFRP sheets and evaluate the strengthening effects of applying different thicknesses and widths of CFRP sheets.
CHAPTER 2 LITERATURE REVIEW

PHC slabs are very widely used in civil engineering applications. Thus, many experimental studies have been conducted to investigate the behaviors of PHC slabs including its flexural and shear capacity. This chapter will review exiting studies on PHC slabs.

2.1 Shear failure types of PHC slabs

Failure types of PHC slabs can be classified in three categories based on different shear span ratio. Shear span ratio is defined as $\frac{a}{d}$, where $a$ is the distance between the support at the loading end and the loading point and $d$ is the specimen depth. According to MacGregor and Bartlett (2000), failure mode can be divided into three main categories based on the shear span ratio. With a smaller shear span ratio ($0 \leq \frac{a}{d} \leq 1.0$), the loading point will be closer to the support which may cause anchorage failure. However, with a larger shear span ratio ($\frac{a}{d} \geq 6.0$), flexural failure would occur. Shear failure is more likely to happen if the range of shear span ratio is between 1.0 and 6.0. In addition, based of EN 1168-08 (Deutsche, 2008), a shear span length of $a = 2.5d$ or $a = 600$ mm, whichever is greater, should be applied during shear test of PHC slabs.

Generally, there are two types of shear failure for PHC slabs, i.e. the flexural shear failure and the web shear failure. According to Araujo et al (2011), flexural shear cracks may occur in sections of which flexural cracks exist and web shear failure is possible to happen in the region close to support without any flexural cracks.
2.1.1 Flexural-shear failure of PHC slabs

![Diagram of flexural-shear failure](image)

Figure 2.1 Flexure-shear failure of PHC slabs (Araujo et al, 2011)

Flexural-shear failure (Figure 2.1) is most likely to occur in sections having bending cracks which is close to the ends of prestressed concrete members. When flexural stress is over the tensile strength of concrete, vertical tensile cracks would initiate at the bottom face of the slab. The crack propagates through the cover till it hits the prestressing strands. With the increase of load, the crack not only becomes wider but also extends across the prestressed reinforcements until reaches approximately the mid-height of the slab. With further increase of load, principal stress in the un-cracked part of the slab could exceed the tensile strength of concrete. Then, the vertical crack develops into a diagonal shear crack. If the originally initiated flexural crack occurs within the transmission length, it may lead to slippage of strands at the end of slab and cause brittle failure immediately (Araujo et al, 2011). Flexural-shear failure is more likely to occur in prestressed concrete members subjected to uniform load than subjected to concentrated load (CTE-73-B4, 1973).
2.1.2 Web shear failure of PHC Slabs

Web shear failure (Figure 2.2) is caused by a diagonal crack in the web of a PHC slab at a location between the support and the loading point. This type of failure occurs when the maximum principle stress results from the combined effect of shear and bending exceeds the tensile strength of concrete (Yang, 1994). Before failure, no crack could be observed in the failure zone. When the resultant shear stress exceeds the tensile strength of concrete, a diagonal crack appears suddenly which leads to total collapse of the PHC slab. The orientation of the crack line is a $90^\circ$ with respect to the direction of the principle tensile stress. Web shear failure is also called “shear-tension” failure. This type of shear failure is more likely to occur in members with high flexural resistance capacity, i.e. high prestressed deep beams or PHC slabs, when subjected to concentrated load close to support.

2.1.3 Factors affecting PHC slab shear capacity

According to studies by Anderson (1976) and Palmer and Schultz (2010), a number of factors could affect the shear capacity of PHC slabs. The main ones include the axial stress level, the geometry of cross section, the concrete strength and the shear span ratio.
2.1.3.1 Axial stress caused by prestressing

Level of prestressing can affect the shear capacity of prestressed members. Shear capacity of prestressed member would increase with higher prestressing level. Because compressive stress provided by prestressing strands could help the concrete to resist the tensile stress due to external force which means that the compressive stress from prestressing would help to reduce the crack width and thus improve the concrete shear strength. However, Yang (1994) developed a 3D finite element model (FEM) to study shear behavior of a simply-supported I-shaped single web slab and found that higher prestressing level might not necessarily help to enhance the shear capacity. In addition, results showed that compared to circular voids, the shear capacity of a slab having noncircular voids would decrease if the prestressing level keeps on increasing.

2.1.3.2 Geometry of cross section

According to requirement of shear design standard provided by ACI 318-08 (ACI Committee 318, 2011), shear reinforcement should be applied to hollow-core units if the total untopped depth of the slab exceeds 317.5mm and the ultimate load levels less than expected.

The geometric shape of slab voids can influence shear capacity of PHC slabs. Yang (1994) concluded that shear capacity of slabs with non-circular voids could be more critical than those with circular voids. The critical shear point of the slab with non-circular voids moved closer to the support at loading end than the slab with circular voids. Thus, the transferred effective prestressing force in strands would be very small.
which may cause the slab to fail at lower load.

2.1.3.3 Concrete strength

Concrete strength has the most impact on the shear capacity of reinforced concrete members. Shear strength provided by concrete is \( V_c = \phi_c \beta \lambda \sqrt{f_c'} b_w d_v \) (CAN/CSA-A23.3-04, 2004), where \( V_c \) is the nominal shear strength provided by concrete, \( \phi_c \) is concrete strength reduction, \( \beta \) is the ability of cracked concrete to transmit shear by aggregate interlock, \( \lambda \) is the modification factor reflecting the reduced mechanical properties of lightweight concrete, \( f_c' \) is the compressive strength of concrete, \( b_w \) is the smallest width of cross section and \( d_v \) is the effective shear depth. This Equation shows that contribution of concrete to shear capacity of PHC slab depends on cross sectional area and concrete compressive strength.

2.2 Carbon fiber reinforced polymer (CFRP) composite

CFRP is a type of strong and light composite material which is widely used in engineering application. This material was invented 50 years ago and it has been applied in civil engineering for 20 years. Mostly, this type of composite material is used to strengthen or rehabilitate decks, beams, columns and slabs of existing civil structures like buildings and bridges.

Carbon fiber reinforced polymer consists of countless filaments of pure carbon. The thickness of CFRP is only 1/10 of width of a human hair. Many studies on applying CFRP in civil engineering structures have been conducted because of its light weight, good
corrosion resistance, flexible formability, high strength and high modulus (Cree et al, 2014). The main advantages of CFRP composite are listed below:

2.2.1 High strength to weight ratio

The mass density of CFRP is only 1/5 of steel but the strength is 8 to 10 times of steel. A carbon fabric sheet is very thin. Thus, when used for repairing structures, it will hardly increase the self-weight of structures (Wang, 2010).

2.2.2 Good durability

CFRP is a non-corrodible material. It doesn’t rust, corrode or rot, and they resist attack from most industrial and household chemicals (Karbhari, 2009). On the other hand, the coefficient of thermal expansion of CFRP is very low (ACP, 2014).

2.2.3 Good fatigue strength

For normal metals, their fatigue strength is 45% of their tensile strength. However, for CRFP, its fatigue strength is 75% of its tensile strength (Wang, 2010).

2.2.4 Ease of construction

CFRP sheets can be easily cut into desired shape and size. It can be applied to structures having different shape and would hardly affect their original geometry. Furthermore, no large machinery is needed to install CFRP composite sheets (Wang, 2010).
2.3 Existing studies of using FRP to strengthen concrete structures

2.3.1 Shear strengthening of Reinforced Concrete beams using externally bonded FRP systems

Currently, many researches have been conducted by using FRP sheets to strengthening shear capacity of RC beams. An experimental study on shear strengthening of RC members using CFRP composite sheets was conducted by Yu (2002). Results showed that applying continuous full FRP wrap was more effective in improving shear capacity of RC beams than applying the interval FRP wrap. Mofidi et al. (2011) evaluated the effects of strip-width-to-strip-spacing ratio when applying externally bonded FRP strips to strengthen the shear capacity of RC beams. He concluded that given a strengthened beam and the CFRP strip-width-to-strips-pacing ratio, beams strengthened by wider CFRP strips could provide more shear resistance than beams strengthened by narrower strips.

Furthermore, a study by Monti and Liotta (2007) showed the type of U-jacketing (bottom and side bonding) of FRP sheets was more effective in enhancing shear resistance of RC beams than the side bonding alone. For specimens strengthened by U-Jacketing FRP sheets, using diagonal FRP strips could better enhance the shear carrying capacity of RC beams than using the vertical strips. It is worth mentioning that the specimen bonded by diagonal (45°) U-Jacketing FRP strips had an enhancement of shear capacity about 92%. Qu (2008) compared the shear behavior between specimens strengthened by full wrapped FRP sheets and U-wrapped FRP sheets through an
experimental study. A shear-span ratio of 2.0 was used for all specimens. Results showed an improvement of shear capacity of 24% in U-wrapped specimens and 36% in full wrapped specimens. All specimens failed in debonding failure. Compared to the U-wrapped specimen, the debonded full wrapped FRP sheets could still resist shear force after the load reached the peak value. The full wrapped specimen eventually failed due to the rupture failure of the FRP wrap close to the support.

The formation of applied FRP sheets could also affect the effectiveness of shear strengthening of RC beams. An experimental study of using mineral-based composite to strengthen shear capacity of concrete structures was conducted by Blanksvärd et al. in 2009. The maximum improvement of shear capacity on C40 (concrete quality based on Eurocode) beam specimen strengthened by CFRP gird was about 100%. On the other hand, two fiber orientations (vertical and horizontal) were also investigated during the experiments. The result showed that the specimens strengthened by the fiber of CFRP sheets oriented in vertical direction had more enhancement of shear capacity than horizontal fiber direction of CFRP sheets.

Additionally, Islam et al (2002) conducted a study to explore shear strengthening of reinforced concrete (RC) deep beams using externally bonded CFRP system. Three different types of CFRP systems were applied to strengthen RC deep beams which include fiber wrap (Figure 2.3(a)), strips (Figure 2.3(b)) and grids (Figure 2.3(c)). Based on the experimental results, it was found that the grid type FRP in normal orientation
performed better than the wrap or the strip type. Furthermore, the shear capacity of RC deep beam was found to improve with the increase of the grid bar cross-sectional area. The specimen strengthened by grid with groove (Figure 2.3(c)) showed about 40% of shear capacity enhancement, which implied that this technology is very effective to enhance the shear capacity of concrete member.

![Figure 2.3 Arrangement of externally bonded FRP systems](image)

Abdel-Jaber et al (2007) conducted an experimental study to investigate the shear behavior of RC beams strengthened by CFRP sheets in different configurations and quantities. Three different fiber orientation types (horizontal, vertical and 45°) of CFRP sheets were attached to the side of each specimen. It was reported that the specimens strengthened by CFRP sheets with fiber oriented in horizontal and vertical directions had similar improvement of shear carrying capacity about 20%. However, specimens
strengthened by CFRP sheets with fiber oriented at 45° with respect to the horizontal direction showed a little bit less enhancement about 10%.

However, results from some experimental studies also showed a remarkable improvement on shear resistance of RC beams to arrange the fiber of FRP sheets at 45° with respect to the horizontal direction. Bukhari et al. (2010) reported that the RC beam strengthened by diagonal CFRP sheets (Figure 2.4) had a significant improvement of 98% on shear carrying capacity. Additionally, it was found by Sim et al. (2005) that the strengthening effect could increase by more than 10% in the specimens strengthened by a 45° fiber orientation than the specimens with a 90° fiber orientation.

![Figure 2.4 RC beam strengthened by side diagonal FRP sheets (Bukhari et al, 2010)](image)

Alzate (2012) indicated that applying different quantity of FRP sheets would cause different contribution of shear resistance of RC beams. Two different thickness of FRP sheets (0.293 mm and 0.176 mm) were investigated in the experimental study. It was found that the average shear capacity of the specimens strengthened by U-jacketing FRP sheets with a thickness of 0.176 mm was 219.5 kN. Besides, applying thicker FRP sheets would help the average of shear capacity of RC beams increase to 241.5 kN.

### 2.3.2 Concrete slabs strengthened by FRP composite

A study on strengthening flexural capacity and ductility of PHC slabs using FRP was
conducted by Liu et al. (2008). A total of seven specimens were tested. The studied parameters included the area of strengthening zone and the thickness of FRP sheets. It was observed that the control slab failed due to rupture of pretressed strand at mid-span, whereas shear failure at support and debonding failure of FRP sheets occurred in all the strengthened specimens. The shear capacity of the strengthened specimens improved from 48% to 109%. Meanwhile, FRP sheets helped to increase the vertical deflection at mid-span after the yielding of prestressed strands.

An experimental study of flexural behavior of PHC slabs externally bonded by CFRP sheets was conducted by Li et al. (2014). It was found that the average improvement of flexural capacity of the strengthened PHC slabs was about 67%. On the other hand, the average deflection of the strengthened specimens at mid-span was doubled.

Florut et al. (2010) conducted a study on attaching externally bonded FRP strips to the bottom for retrofitting two-way RC slabs. The specimens were subjected to a uniformly distributed load over a central patch. Compared to the control slab, the specimens bonded by FRP strips showed about 59% increment of maximum load capacity and considerable increase of mid-span deflection.

Another test of using hybrid composite material to reinforce flexural capacity of RC slab was investigated by Mosallam et al. (2012). The hybrid composite system included high performance concrete and CFRP composite sheets. This system was installed on the
top surface of the floor slabs or bridge decks. Results showed that the maximum improvement on the flexural capacity was 164% and the maximum increase of the deflection at the mid-span was 122%. The failure of strengthened specimens was commonly dictated by horizontal shear failure and debonding of the overlay from the slab close to the intermediate support.

2.3.3 Summary

It can be seen from the above review that most studies reported considerable improvement of shear or flexural capacity of RC members strengthened by FRP composite sheets. Several parameters could affect the performance of FRP strengthened concrete members. They include

1. Bonding types of FRP sheets

The bonding types of FRP sheets used in existing studies are shown in Figure 2.5.

Figure 2.5 Bonding types of FRP sheets applied in strengthening concrete members
2. Orientation of FRP fiber

Mostly, the fibers of FRP were arranged to be perpendicular to the member longitudinal axis. In some cases, FRP sheets were applied with fiber oriented at 45° with respect to member axis. Most studies reported that specimens strengthened by FRP sheets with fiber orientation of 90° had more improvement in the ultimate capacity than those strengthened with 45° FRP fiber orientation.

3. Quantity of FRP sheet

The applied quantity of FRP sheets could also affect the improvement of load carrying capacity of strengthened concrete members. The strengthening effect of FRP composite mostly related to the stress level in FRP before failure.

2.4 Shear Strengthening of Single Web Prestressed Hollow Core Slabs Using Externally Bonded FRP Sheets

Currently, no study is available to evaluate the feasibility and effectiveness of using CFRP composite sheets to strengthen shear capacity of PHC slabs. Based on the existing studies of installing FRP composite to enhance the performance of RC members, several parameters, i.e. the prestressing level of PHC slab, the length of strengthening zone and the thickness of FRP sheets, should be included in this study. The entire project of studying the feasibility and effectiveness of using externally bonded CFRP composite sheet to strengthen PHC slabs consists of two phases. The work in Phase I focused on applying the proposed shear strengthening technique to I-shaped single-web beams cut.
out longitudinally from full-width PHC slabs to assess the feasibility and effectiveness of this shear strengthening technique. It was conducted by a former master student (Wu, 2015). The proposed study is the second phase of the project, of which the shear strengthening technique will be applied to full-width PHC slabs.

Figure 2.6 shows the proposed shear strengthening technique applied to the I-shaped single-web PHC slab specimen. A total of sixteen I-shaped single-web specimens were tested the Phases I, which included eight W1-Series (1-prestressing strand web beams) and eight W2-Series (2-prestressing strand web beams).

Figure 2.6 Proposed shear strengthening techniques for single web hollow core slabs (Wu, 2015)

All specimens were tested over a simply supported clear span of 4499 mm and a shear span ratio \( a/d \) of 2.0 was used for W1-Series specimens whereas a shear span ratio of 2.5 was used for W2-Series specimens.

Results obtained from the first phase indicated that shear capacity of the strengthened single-web PHC slab specimens can be enhanced by increasing the thickness of CFRP sheets and the length of the strengthened zone. It was found that the single-web beam specimens strengthened by 2 layers CFRP sheets on each side can not
only improve the shear capacity but also enhance the ductility before failure. The average improvements on shear capacities of W1-series and W2-series specimens were 14.50% and 27.25%, respectively. Thus, it was expected that the proposed technique would also be effective when applied to the full-width hollow core slabs.

2.5 Numerical Study on Shear Strengthening of Reinforced Concrete Members Using FRP composite sheets

Finite element modelling is an effective method to analyze the complex behavior of many kinds of structures. Although experimental data is valuable in understanding the behavior of structural members, analytical and numerical solutions are needed to further comprehend the associated failure mechanisms and are flexible in parametric study (Coronado et al., 2006). Currently, extensive researches have been conducted to simulate the behavior of FRP-strengthened concrete structures and one of the most popular software used for this propose is ABAQUS.

Coronado et al (2006) conducted experimental study of reinforced concrete (RC) beams strengthened with FRP laminates. Subsequently, using ABAQUS to evaluate the effects of the concrete constitutive behaviors and different modeling considerations. A plastic-damage model was used in order to describe the constitutive behavior of concrete. In addition, the constitutive relationship of steel was modelled as elastic-perfectly plastic and that of FRP was modelled as linear elastic. The four-node bilinear strain element was used to model the concrete. Both of steel and FRP were simulated by the two-node truss
element. The interfacial relationship between the FRP sheets and the concrete was assumed to be perfect bonding. It is worth mentioning that the result predicted by the FE model without the simulation of epoxy only showed a slight difference compared to the experimental results. This implies that the modeling of epoxy has a minor effect on the overall response of the FRP strengthened RC beams.

Another study that used both the experimental and numerical approaches was by Ebead et al. (2002), of which the effectiveness of using CFRP strips and GFRP sheets to strengthen the flexural capacity of two-way RC slabs was evaluated. In the finite element analysis, the concrete panel was modelled by the eight-node quadrilateral shear-flexible thick-shell elements which had six degrees-of-freedom per node. Both of FRP and steel were modelled as rebar in concrete. The steel reinforcement and the FRP material were assumed to be perfectly bonded with the surrounding concrete. The maximum error between the experimental and the finite element simulation results was reported to be only 1.06%. Besides, a parametric study was also carried out to analyze the strengthening effectiveness related to the strengthening area, the span of the slab, the reinforcement ratio, and the thickness of the slab.

El Sayed et al. (2005) conducted a non-linear finite element modelling of flexure failure for FRP-strengthened two-way slabs. The concrete was simulated by the solid element and the truss element was used for steel reinforcement. The thin shell element was used to represent both the unidirectional and bidirectional FRP sheets. A smeared
crack model was used to simulate the tensile behavior of the cracked concrete. A linear-elastic model was used to simulate the behavior of FRP composite. The main findings from this study indicated that the failure of the strengthened slabs was mainly caused by debonding between the concrete and the FRP composites. The interface between the FRP composite and the concrete was simulated by the nonlinear translational spring element which only had stiffness in the horizontal direction. A force–displacement relationship was developed based on the bond–slip model used. The proposed model could capture the debonding phenomenon and predict possible failure modes associated with the delamination of the FRP composites.

Chen et al. (2012) discussed several issues in the finite element modelling of RC beams shear-strengthened by externally bonded FRP sheets. The concrete was modelled by the four-node bilinear plane stress element. The truss element was used to simulate both strands and FRP. The results were mainly related to the modelling of interface between the concrete and the steel, and that between the concrete and the FRP. The authors reported that the assumption of a perfect bond between the concrete and the FRP might over-estimate the shear capacity of the beam. A stronger bonding between steel tension bars and concrete could affect the shear capacity of RC beams, the number of diagonal cracks and the angle of main shear crack. In conclusion, how properly the bonding behavior at each interface is modelled would significantly affect the numerical simulation of shear behavior of RC beams strengthened by FRP.
The details of the finite element models in the above reviewed literature are summarized in Table 2.1. There are several aspects that could affect the simulation accuracy of the behavior of FRP-strengthened RC members. They are:

1. The constitutive model, element type and material properties of concrete, steel and FRP,

2. The interfacial relationship between concrete and steel as well as concrete and FRP.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Structural Member</th>
<th>Constitutive Relation</th>
<th>Element Type</th>
<th>Interfacial Relation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coronado et al. (2006)</td>
<td>RC beams</td>
<td>damaged plasticity</td>
<td>CPE4R</td>
<td>truss</td>
</tr>
<tr>
<td>Ebead et al. (2002)</td>
<td>RC slab</td>
<td></td>
<td>thick-shell</td>
<td>rebar</td>
</tr>
<tr>
<td>El Sayed et al. (2005)</td>
<td>RC slabs</td>
<td>smeared crack</td>
<td>solid</td>
<td>truss</td>
</tr>
<tr>
<td>Chen et al. (2012)</td>
<td>RC beams</td>
<td></td>
<td>CPS4</td>
<td>truss</td>
</tr>
</tbody>
</table>

(CPE4R: 4-node bilinear strain element, CPS4: 4-node bilinear plane stress element.)
CHAPTER 3 EXPERIMENTAL STUDY

Experimental study is an important and effective approach to understand the behavior of structural members subjected to different types of loads. In the current work, an experimental study, phase II of the project, are conducted on full-width PHC slabs to investigate its shear behavior when externally bonded by CFRP sheets. The specimens were tested in the Structures lab at the University of Windsor.

3.1 Test specimens

A total of ten full size PHC slabs specimens are tested in this phase. All specimens have a length of 4575mm, a depth of 305mm and a width of 1219mm. The test specimens were divided into two series, S1 and S2, based on the prestressing level. Each S1-series (low-prestressing) specimen has six longitudinal prestressed strands (one strands per web) with a diameter of 13 mm at the bottom of the slab. On the other hand, each S2-Series specimen (medium-prestressing) has eight longitudinal strands (two strands per each of three middle webs) with the same diameter as those in S1-Series. There are four S1-series specimens and six S2-series specimens. The cross sections of the specimens in each series are shown respectively in Figures 3.1(a) and 3.1(b).

The specifications of each specimen are listed in Table 3.1. Taking S2-series specimens as an example, one of them was used as the control specimen, S2-C, with no CFRP sheets applied. “S” represents slab specimen, “2” represents medium prestressing level (8 prestressing strands) and “C” represents control slab. Two different thicknesses of
applied CFRP sheets, i.e. 1 layer and 2 layers on each slab void surface respectively, are investigated during the experimental study. Future more, the effect of increasing strengthened zone length from 300 to 450 was investigated. The ID of the strengthened specimens is defined to include the thickness and the length of the strengthened zone. For example, “S1-2-450” represents 2 layers of CFRP sheets with 450 mm length are installed for a PHC slab with low prestressing level (6 prestressing strands).

![Diagram](image)

(a) S1-series (low prestressing) slab (unit: mm)

(b) S2-series (medium prestressing) slab (unit: mm)

Figure 3.1 Cross section of each type of specimen
Table 3.1 Parameters of PHC slabs used in experimental study

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>No. of CFRP sheet layer</th>
<th>Strengthened zone length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-C</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>S1-1-450</td>
<td>1</td>
<td>450</td>
</tr>
<tr>
<td>S1-2-450</td>
<td>2</td>
<td>450</td>
</tr>
<tr>
<td>S1-2-450-2nd</td>
<td>2</td>
<td>450</td>
</tr>
<tr>
<td>S2-C</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>S2-1-450</td>
<td>1</td>
<td>450</td>
</tr>
<tr>
<td>S2-2-300</td>
<td>2</td>
<td>300</td>
</tr>
<tr>
<td>S2-2-450</td>
<td>2</td>
<td>450</td>
</tr>
<tr>
<td>S2-2-450-2nd</td>
<td>2</td>
<td>450</td>
</tr>
<tr>
<td>S2-2-450-3rd</td>
<td>2</td>
<td>450</td>
</tr>
</tbody>
</table>

All specimens were fabricated at the supplier site, the Prestressed System Inc., and shipped to the Structurers Lab at the University of Windsor for testing. The surface of each slab void where CFRP sheets would be attached to was first cleaned and polished by steel brush. The uni-directional CFRP sheets were then bonded to the void surface in the circumferential direction by applying a wet layup process. The surface of the slab voids and the CFRP sheets should be saturated by epoxy resin before directly bonding the sheets to the void surface. The direction of CFRP fiber was perpendicular to the longitudinal axis of the slab specimen (Figure 3.2). The specimens would be tested after at least two days after attaching the CFRP sheets.
3.2 Experimental setup

The experimental setup is shown schematically in Figure 3.3. All specimens were tested over a simply supported clear span of 4499 mm with the bearing plate width of each support being 76 mm. The support near the loading point was a hinge and at the other side was a roller. This would ensure that no axial force would be generated during test. The shear span to depth ratio (a/d) of all slab specimens was 2.5, i.e. the concentrated load was located at a distance of 762.5 mm from the center of hinged support.
3.3 Instrumentations and Testing Procedures

All specimens were instrumented at the loaded end. As shown in Figure 3.4(b), two linear variable displacement transducers (LVDT) were used, one was for measuring the end slippage of the prestressed steel in the second web, and the other was used to measure the vertical displacement at the loading point. Additionally, three displacement transducers, Pi-gauges, were installed to measure the strain in the longitudinal direction of the PHC slabs along the center line of the first, the second and the middle web in the shear-tension region to measure the tensile strain of concrete or CFRP sheets (Figure 3.4(b)). According to the results from Phase I (Wu, 2015), the location of each Pi-gauge was set at the intersection between the specimen horizontal center line and the line...
connecting the inner edge of the support bearing plate and the inner edge of the loading plate (Figure 3.4(a)). The forth Pi-gauge was placed on the slab top surface in the longitudinal direction to measure the concrete compressive strain near the loading point. Besides, three electrical foil strain gauges were glued on the surface of the CFRP sheets on the opposite side of the Pi-gauge to measure the tensile strain in the CFRP sheets.

Based on Deutsch Norm (2008), the loading process consists of two steps. During the test, the specimen was first loaded to 70% of the predicted failure load and then unloaded. In the second step, the slab was re-loaded until failure by displacement control. Figure 3.5 shows a typical instrumentation arrangement of a slab specimen during test.
3.4 Material properties

3.4.1 Concrete

The test specimens were constructed using the same batch of normal-weight concrete with an average 28-day compressive strength of 60 MPa. A total of thirty $100 \times 200$ mm
and fifteen 150 × 300 mm cylinders were prepared under the same condition to evaluate the compressive and tensile strength of concrete at the test time. The average result of the concrete ultimate compressive strength $f'_c$ obtained from the compression test were 59.3 MPa for S1 specimens (low-prestressing PHC slabs with 6 prestressing strands) and 52.2 MPa for S2 specimens (medium-prestressing PHC slabs with 8 prestressing strands). The Young’s modulus $E_c$ was taken as 22167 MPa for the S1 specimens and 23946 MPa for the S2 specimens, respectively.

3.4.2 CFRP

SikaWrap-900C CFRP sheets and Sikadur300 Epoxy were used in strengthening the PHC slab specimens. The SikaWrap-900C is a uni-directional, fleece stabilized, stitched and heavy carbon fiber fabric for the wet application process of structural strengthening (SIKA Canada). The mechanical properties of the CFRP sheets with epoxy resin are either provided by the supplier or determined by tensile tests on representative samples in accordance with ASTM D3039 (ASTM D3039/D3039M, 2014). The results showed a linear-elastic stress-strain relationship with average values of elastic modulus and tensile strength being 100 GPa and 1120 MPa, respectively (SIKA Canada).

3.4.3 Prestressing strands

The prestressing strands are 7-wire, low-relaxation strands which has a diameter of 13 mm and an ultimate tensile strength of 1860 MPa.
3.5 Results of S1-series

3.5.1 Experimental observation

One of the low-prestressing specimen without installing any CFRP sheets, S1-C, was used as the control slab. A typical shear-tension failure occurred suddenly on S1-C when the load reached 291 kN. The crack initiated at a distance of 74 mm from the inner edge of the hinge support and extended up to the middle of the loading plate. The horizontal length of the crack was 635 mm which covered the entire shear-tension region. The width of the crack measured after failure was 2 mm. It’s worth mentioning that the crack just passed through the middle of the first Pi-gauge, as shown in Figure 3.6(a), which proved that the arranged Pi-gauges and strain-gauges could efficiently measure the critical strain behavior during test.

Both Figures 3.6(b) and 3.6(c) show the crack pattern of S1-1-450 specimen. A typical shear-tension failure occurred at the load of 356 kN. The shear crack was overserved from the inner edge of the support and stopped at the top of the slab with a horizontal distance of 385 mm and an angle of 44° with respect to the horizontal. The crack also propagated across the top surface of the slab and another longitudinal crack developed suddenly on top surface at mid-point of the first void. As a result, a piece of concrete spalled off immediately at the failure load (shown in Figure 3.6(c)).

Figure 3.6(d) and (e) showed the crack patterns of S1-2-450 and S1-2-450-2nd, respectively. The S1-2-450 specimen failed at 332kN in a typical shear-tension failure.
mode. The crack initiated from the inner edge of the hinge support and stopped at the top of the slab near the loading point. The horizontal length of the crack was 520 mm with an angle of 34° respect to the member axis. Similarly, a typical shear failure of the S1-450-2nd specimen occurred when the load reached 338 kN. Although the shear crack of S1-1-450-2nd showed the similar growth trend (39°) of the first S1-2-450 specimen, the initiation of the crack moved 166 mm closer to the loading point and the horizontal length increased from 520 mm (the first S1-2-450 specimen) to 619 mm.

Besides, a fine vertical flexural crack near the loading point was observed on all strengthened S1-series specimens when the load reached around 300 kN. However, after the final failure, the flexural crack closed and was no longer visible due to the prestressing force. The flexure cracking loads of the S1 and S2-series specimens were calculated according to the PCI Manual for the Design of Hollow Core Slabs (1998). The cracking load of S1-series is 290 kN while it is 400 kN for S2-series. This explains the appearance of the vertical flexural cracks in S1-series.
Moreover, vertical and radial cracks were also observed around the prestressing strands (shown in Figure 3.7) during testing for all S1 and S2 specimens. This would
mainly increase the strand slippage and affect the overall capacity of test specimens.

![S1-C](image)

(a) S1-C

![S1-1-450](image)

(b) S1-1-450

Figure 3.7 Cracking patterns of S1-series around the strands at loading end

Although only S1-1-450 specimen showed instantaneous concrete spall off at the failure load, no debonding was observed between the CFRP sheets and the concrete before failure on all strengthened S1-series specimens.

### 3.5.2 Ultimate load

The results of the ultimate load of S1-series PHC slab specimens are summarized in Table 3.2. The shear capacity of the control slab, S1-C, was 291 kN. With a considerable
enhancement of 22%, the failure load of S1-1-450 was 356 kN. The average ultimate load of the two specimens strengthened by 2 layers CFRP sheets, S1-2-450, was 335 kN, which had an increment of 15% compared to the control slab. It should be noted that, the specimen strengthened by 2 layers CFRP sheets showed lower enhancement on shear capacity of PHC slabs than that strengthened by 1 layer CFRP sheets. This could be due to the material property of the PHC slab itself which will be explained by the discussion of Figure 3.9.

Table 3.2 Test results of S1-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>S1-C</td>
<td>291</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S1-1-450</td>
<td>356</td>
<td>22</td>
<td>Brittle</td>
</tr>
<tr>
<td>S1-2-450</td>
<td>332</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>S1-2-450-2nd</td>
<td>338</td>
<td>16</td>
<td></td>
</tr>
</tbody>
</table>

3.5.3 Load-deflection behavior

Figure 3.8 shows the load-deflection curves of S1-series specimens. As shown in the figure, all specimens had a similar behavior during the elastic range. The strengthened specimens showed higher shear capacity and much larger deformation than the control specimen. It is interesting to see that the S1-1-450 specimen showed a bi-linear behavior before failure. During the first elastic range, S1-1-450 showed a very similar behavior as that of the S1-C, i.e. similar peak load (291 kN for S1-C and 280 kN for S1-1-450) and similar vertical deflection at the loading point (2.7 mm for S1-C and 3.3 mm for S1-1-450). However, with the strengthening provided by 1 layer 450 mm long CFRP
sheet, the strengthened specimen was still capable to resist more shear force beyond the elastic range and exhibited larger deflection. Finally, the S1-1-450 specimen failed at an ultimate load of 356 kN and the maximum deflection of this specimen was tripled (10 mm) when compared to the control slab.

The S1-2-450-series specimens also showed similar behavior during elastic range compared to the control specimens. The average improvement of the ultimate load in S1-2-450-series specimens was 15%. It is clear to see from the plot that when the load of both S1-2-450-series specimens reached the ultimate load of the control slab around 290 kN, the slope of load-deflection curve decreased. Both S1-1-450 and S1-2-450 specimens showed stiffness loss after 290 kN (decrease in the slope of the curve). This could be due to the slab reached the shear capacity provided by concrete and more cracks were being developed. However, the existence of CFRP sheets helps carry more shear force and increase the strength beyond 290 kN.
Figure 3.8 Load-deflection behavior of S1-series

The strand slippage of the second web at the loading end is shown in Figure 3.9. All specimens showed a linear trend during the entire loading process. However, the S1-1-450 specimen showed lower initial stiffness than the control specimen and a strand slippage of about 2.4 mm which was more than three times of the strand slippage of the non-strengthened specimen. The S1-2-450-series specimens had similar behavior compared to the S1-1-450 specimen. In addition, both S1-2-450 specimens showed consistent results in terms of the slope of the load-deflection curve, the ultimate failure load and the strand slippage (about 2.2 mm).

Figure 3.9 can also explain the reason why the 2 layer-specimens showed lower shear strengthening effect than the 1 layer-specimen. It can be seen that the slippage of
the prestressing strand in S1-2-450 was more considerable than that in S1-1-450 corresponding to the same load. In other words, S1-2-450 had more prestressing loss during the loading process than S1-1-450, which would reduce the shear capacity of the PHC slabs. This slippage was mainly due to the vertical and radial cracks occurred before failure around the strands at the loading end.

Even though the shear strengthening performance of the S1-2-450-seris specimens was not as good as expected, the tested two specimens still showed an average of 15% enhancement on shear capacity of PHC slabs, which indicated that the specimens strengthened by 1 layer and 2 layers CFRP sheets were capable to resist higher shear load.

![Figure 3.9 Load-end slip behavior](image)

3.5.4 Crack pattern

Figure 3.10 shows the crack profile of S1-series specimens at failure. Each failed
specimen exhibited a typical diagonal shear crack. The angle of the cracks with respect to the slab longitudinal axis varied from 27° to 44°. Installing 1 layer of CFRP sheet reduced the effective shear-tension region of S1-1-450 significantly more than 50% from 635 mm in the non-strengthened case, S1-C, to 311 mm. Besides, with the installation of 2 layers CFRP sheets, occurrences of crack in the two S2-2-450-series specimens moved closer to the loading point which results in less angle of crack.
3.5.5 Strain behavior

Figure 3.11 shows the relationship between the load and the longitudinal compressive strain in concrete at the top surface of the slab 565 mm from the loading end measured by Pi-gauge 4 (Figure 3.4). All specimens showed similar trend within the elastic range. However, the S1-1-450 specimen showed larger compressive strain compared to that of the control slab at the failure load. On the other hand, two S1-2-450-series specimens showed similar growth trend compared to the control slab but larger compressive strain at failure. All concrete compressive strains were far below the ultimate compressive strain corresponding to concrete crushing, i.e. 3500 microstrain,
which implies the failure was mainly caused by shear.

Figure 3.1 shows the relationship between the load and the longitudinal tensile strain of CFRP sheets (along the circumferential direction of the slab voids) measured by strain gauge 1 at the first web of the PHC slab (Figure 3.4(b)). The curve of S1-1-450 showed an elastic range from the beginning until the load reached around 290 kN where the concrete was resisting all the load. Relying on the tensile strength provided by the CFRP sheets, the slab could still resist the shear force increment and further deflect until the final shear failure. The slope of the second range corresponding to the load-strain curve of S1-1-450 showed a slight decrease compared to the first range. Because the shear resistance was mainly provided by the concrete during the first range, around 290
kN load, a shear crack was initiated and the load was transferred to the CFRP sheets as indicated by the rapid and sudden increase in the strains. The load-strain curve of both the S1-2-450-series specimens exhibited a similar behavior within the elastic range. The linear trend of the curve indicated that no crack occurred before the final shear-tension failure. However, S1-2-450 failed to increase in the ultimate shear strength due to the excessive strand slippage as explained before. Therefore, there was no noticeable increase in the strain of CFRP sheets after cracking.

![Load vs longitudinal tensile strain in CFRP sheets relationships](image)

**Figure 3.12** Load vs longitudinal tensile strain in CFRP sheets relationships of S1-series specimens

### 3.6 Results of S2-series

#### 3.6.1 Experimental observation

The control specimen, S2-C, showed typical brittle shear failure. When the load reached 280.5 kN, shear failure occurred suddenly along with a big noise. An inclined
crack initiated from the support, propagated through the slab and ended near the loading plate at the top. The crack covered the entire shear-tension region with a horizontal distance of 731 mm from the support end. The maximum width of this critical shear crack was 2 mm. Thus, 280.5 kN was considered as the ultimate shear capacity of the S2-series control slab. Figure 3.13(a) shows the crack pattern of the control specimen.

Figure 3.13(b) shows the crack pattern of the S2-2-300 specimen. A typical shear-tension failure was observed when the load increased to 310 kN. The crack initiated at a distance of 455 mm away from the end of slab, propagated upwards at an inclination angle of 37° and stopped near the outer edge of the loading plate. This result indicated that the shear resistance of the critical shear-tension region was considerably enhanced which led the shear crack occurred out of the strengthened zone. The horizontal length of the crack was 527 mm with a crack width of 2 mm.

The crack pattern of the S2-1-450 specimen, shown in Figure 3.13(c), is similar to that of the control slab. The shear failure occurred suddenly at 328 kN. A critical shear crack initiated from the inner edge of the support bearing plate and extended to the location near the loading plate. The horizontal length of the crack was 567 mm and the maximum crack width was 2 mm.

Shear failure of S2-2-450 (shown in Figure 3.13(d)) occurred when the load increased to 387 kN. A critical shear crack also initiated at the slab bottom and propagated through the slab height with an inclination angle of 44° with respect to the horizontal. It is
interesting to note that the crack started from a distance of 224 mm away from the inner edge of the bearing plate and then extended to the inner end of the loading plate. It covered a longitudinal region of 530 mm with the maximum crack width of 3 mm. Results of S2-2-450 showed that if the specimen is strengthened by 2 layers of 450 mm long CFRP sheets, the ultimate load could improve significantly by 38%.

Because S2-2-450 manifested considerable enhancement on shear capacity, two more tests were repeated to verify its strengthening effect. These two specimens were named as “S2-2-450-2nd” and “S2-2-450-3rd”, respectively. The crack patterns of these two specimens are shown in Figure 3.13(e) and (f), respectively. Brittle shear failure in the S2-2-450-2nd specimen occurred at a load of 368 kN, which was lower than the first S2-2-450 specimen under the same strengthening condition. This was believed to be caused by the local debonding of the CFRP sheets in one of the slab voids before the test, as shown in Figure 3.14. The cracking pattern of the S2-2-450-2nd specimen showed similar growth trend as the first S2-2-450 specimen. However, the location and the length of the crack were similar to the S2-C specimen. On the other hand, the S2-2-450-3rd specimen showed more consistent ultimate shear capacity of 378 kN as compared to the first S2-2-450 specimen. A shorter crack with a horizontal length of 342 mm was observed after the failure. In addition, the crack initiated very close to the inner edge of the hinged support and stopped at the half distance between the support and the loading point. This could be possibly caused by the coarse aggregate in the concrete which might
change the prorogation path of the crack.

(a) S2-C

(b) S2-2-300 specimen

(c) S2-1-450 specimen

(d) S2-2-450 specimen
Vertical and radial cracks around pretressing strand at the loading end can be observed on all strengthened specimens after failure (Figure 3.15). This was caused by the high prestressing force in the strand at the end of slab, as explained earlier.
(a) S2-1-450

(b) S2-2-450

(c) S2-2-450-2nd
In the S2-series specimen, the critical shear cracks grew through the slab and no debonding was observed between the CFRP sheets and the concrete surface before failure.

### 3.6.2 Ultimate load

The test results of the all S2-series specimens are summarized in Table 3.3. All specimens failed in a brittle shear failure mode. The shear failure of control slab occurred at 280.5 kN. The ultimate load of S2-1-450 specimen was 328 kN which indicated that the specimen strengthened by 1 layer CFRP sheet with a length of 450 mm had a 17% increment in the ultimate shear capacity. The S2-2-300 specimen failed at 310 kN with less shear capacity enhancement of 10.5%. Whereas, all specimens strengthened by 2 layers CFRP sheets with a length of 450 mm had at least 31% improvement on shear capacity. The average ultimate load of the S2-2-450-series specimens reached 378 kN and gained 34.4% increment in shear capacity. These results not only demonstrate the effectiveness of the proposed shear strengthening technique in full-width PHC slabs, but
also indicated that increasing thickness of CFRP sheets would have a significant impact on improving the shear performance of PHC slabs.

Table 3.3 Test results of 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></td>
</tr>
<tr>
<td>S2-1-450</td>
<td>328</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>S2-2-300</td>
<td>310</td>
<td>11</td>
<td>Brittle</td>
</tr>
<tr>
<td>S2-2-450</td>
<td>387</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-2nd</td>
<td>368</td>
<td>31</td>
<td></td>
</tr>
<tr>
<td>S2-2-450-3rd</td>
<td>378</td>
<td>35</td>
<td></td>
</tr>
</tbody>
</table>

3.6.3 Load-deflection behavior

Figure 3.16 shows the load-deflection relation curves of the S2-C and S2-450-series specimens. All specimens exhibited a similar behavior in the elastic range. The strengthened specimens showed higher shear capacity than the non-strengthened one. The maximum displacement of the S2-1-450 was about 2.4 mm which is roughly the same as the S2-C specimens but at 17% higher ultimate load. However, the S2-2-450 specimens had larger vertical deflection with an average value of 3.8 mm at the same loading position corresponding to the ultimate load. It is worth noting that when the load in all the S2-2-450-series specimens reached approximately the ultimate load of the S2-1-450 specimen, the slope of its load-deflection curve decreased slightly. This could be due to the reason that the slab reached the shear capacity of the concrete and more cracks occurred. With the installation of two layers CFRP sheets, the slab could not only carry
more shear load until to the failure, but also resist more deflection. In addition, although
the sudden drop in the load-deflection curves of all specimens after reaching the ultimate
load indicated that the failure mode of all specimens belong to brittle shear-tension failure,
with the installation of 2 layers of 450 mm long CFRP sheets, the ductility of the
S2-2-450 specimens was considerably enhanced.

The two repeated tests of S2-2-450 also showed similar behavior in the elastic range
to the first S2-2-450 specimen. Nevertheless, there was no sizable improvement on
ductility of these two specimens.

Figure 3.16 Effect of strengthening thickness on the Load-deflection behavior of
S2-450-series

Figure 3.17 compares the load-deflection relationships between the S2-C, S2-2-300
and S2-2-450 specimens, which shows the effect of strengthening length on the shear
performance of PHC slabs. The load-deflection curves of all three specimens had a linear trend within the elastic range up to failure. Compared to the control slab, the ultimate shear capacity of the S2-2-300 and S2-2-450 specimens increased 11% and 38% to 310 kN and 387 kN, respectively. The installation of longer CFRP sheets, S2-2-450, showed 27% more enhanced shear capacity than S2-2-300. This could be due to that the 450 mm long CFRP sheet could better cover the critical shear-tension region than that of 300 mm. Besides, the ductility of both was also improved. This indicated that applying 2 layers CFRP sheets did not only improve the ultimate shear capacity, but also enhanced the ductility of the strengthened PHC slab.

![Graph showing effect of strengthening length on load-deflection behavior](image)

**Figure 3.17** Effect of strengthening length on the load-deflection behavior of S2-2-series

Figure 3.18 shows effect of strengthening thickness on the strand slippage at the loading end. Similar to the load deflection curves, the load-slippage curves of S2-C and
S2-1-450 (shown in Figure 3.18(a)) manifested similar load-slip behavior (about 0.75 mm up to about 300 kN), while the prestressing strand at the loading end of the S2-2-450-series specimen had larger slippage with an average value of approximately 1.2 mm up to 380 kN. This phenomenon suggests that the specimen strengthened by 2 layers of CFRP sheets could resist higher shear load which may also lead to anchorage failure at the slab end (Figure 3.15(b), (c) and (d)).

![Graph showing load-end slip behavior](image)

**Figure 3.18** Effect of strengthening thickness on the load-end slip behavior

### 3.6.4 Crack pattern

Figure 3.19 shows the crack profile of the S2-series specimens at failure. A typical diagonal shear crack was observed in each specimen at the instant of failure. The orientation of the cracks was between 30° and 46° with respect to the specimen longitudinal axis. In all strengthened specimens, the effective shear-tension region was reduced from 596 mm (S2-C) to 567 mm (S2-1-450), 527 mm (S2-2-300) and 500 mm
(average of S2-2-450-series), respectively. The S2-C and S2-1-450 specimen had similar critical shear crack pattern and the difference was that the crack initiating location of S2-1-450 specimen moved closer to the support. Whereas in the case of S2-2-300, the initiation of shear crack moved closer to the loading point by 320 mm compared to the control slab. In S2-2-450-series, the crack of the first tested specimen started at a distance of 224 from the inner edge of the hinge support. But the initiations of the cracks in the other two repeated specimens were closer to the support. Besides, the S2-2-450-2nd specimen showed similar crack pattern as the control and the S2-2-450-3rd specimen had the shortest crack length with its horizontal projection being 342 mm.

Thus, for S2-1-450, installing 1 layer CFRP sheet could not prevent the occurrence of shear crack in the original shear-tension region. However, the second layer of CFRP sheet in S2-2-300 and S2-2-450 added additional shear resistance capacity to the specimen which made it sufficient to improve the shear strength in the original critical shear zone and pushed the formation of the shear crack to be closer to the loading point. However, the shear cracks formed in the two repeated S2-2-450 specimens were still within the original shear-tension region. The reason could be mainly induced by the inhomogeneous property of the concrete.
3.6.5 Strain behavior

Figure 3.20 shows the relationship between the load and the longitudinal compressive strain in concrete at the top surface of the slab measured by Pi-gauge 4 (Figure 3.4). In Figure 3.20(a), all the S2-2-450-series specimens showed the similar
behavior within the elastic range. It should be mentioned that because of the loose of the Pi-gauges during test, the S2-1-450 specimen did not show the compressive strain behavior for the entire loading process. Furthermore, all concrete compressive strains were far below the concrete crushing strain (3500 microstrain). Therefore, the failure was mainly caused by shear-tension failure.

Figure 3.20 Effect of strengthening thickness on the Load vs top longitudinal concrete compressive strain relationships of S2-series specimens

Figure 3.21 shows the relationship between the load and the longitudinal tensile strain of CFRP sheets measured by strain gauge 2 at the second web of the PHC slab (Figure 3.4(b)). As shown in Figure 3.21 (a), both of the S2-1-450 and the S2-2-450-series specimens exhibited a similar linear behavior during the elastic range. When the load reached around the failure load, the CFRP sheets could still resist the load.
as indicated from the rapid increase in the strain. This is a clear indication that there was no debonding failure between the CFRP sheets and the concrete. As the load in the S2-1-450 specimen was lesser than 290 kN, the measured strain of the CFRP was very small and no crack could be observed during this range. However, when the load exceeded 290 kN, the concrete cracked and the CFRP sheets started to resist the shear load until the final failure. However, because of the additional layer of CFRP sheets, the S2-2-450 specimen could resist more shear load than the S2-1-450 specimen and finally failed at around 380 kN. Additionally, these results imply that the role of CFRP sheets can be considered as similar to that of the shear reinforcement. Therefore, bonding 2 layers of CFRP sheets on each side of the slab void is as if more stirrups are arranged in the strengthening zone, which would provide more enhancement to the shear capacity of the S2-2-450-series specimen.

The load vs longitudinal tensile strain in CFRP sheet curves of S2-2-300 and S2-2-450 are shown in Figure 3.21(b). Similarly, no debonding failure occurred in the S2-2-300 specimen during the entire loading process. However, S2-2-300 showed larger tensile strain in the CFRP sheets compared to S2-2-450 corresponding to the same loading level, which could indicated that the longer CFRP sheets could provide more shear resistance to PHC slabs. In other words, it could be assumed the arrangement of shear stirrups covered a longer length of the critical region.
(a) Effect of strengthening length (S2-450-series)

(b) Effect of strengthening length (S2-2-series)

Figure 3.21 Effect of strengthening thickness on the Load vs longitudinal tensile strain in CFRP sheets relationships of S2-series specimens

3.7 Effect of prestressing level

As mentioned in the reviewed literature, higher prestressing level might not
necessarily help to enhance the shear capacity of PHC slabs (Yang, 1994). The comparisons of the load-deflection relation curves between the S1-series and the S2-series are shown in Figure 3.22. The load-deflection curves between the control slabs are shown in Figure 3.22(a), which matched well within the elastic range. However, the low-prestressing specimen showed higher shear capacity (291 kN) than the medium-prestressing specimen (280 kN). This is consistent with the literature finding (Yang, 1994) that higher prestrssing level might not necessarily help to enhance the shear capacity of PHC slabs. Because the main function of the prestressing strands at the bottom of PHC slabs are to resist the tensile stress induced by bending, whereas the shear force is mainly carried by the concrete in the middle of each web. According to the data provided by the supplier, S1-series and S2-series had different concrete strength (59.3MPa for S1-series and 52.2 MPa for S2-series, respectively). It is reasonable that with the higher concrete strength, S1-series showed higher shear capacity than S2-series. In other words, the shear capacity of the non-strengthened PHC slabs highly depends on the tensile strength of the concrete itself.

Figure 3.22(b) shows the load-deflection relation comparison between S1-1-450 and S2-1-450. Again, the low-prestressing specimen shows more enhancement on shear capacity, i.e. 356 kN for S1-1-450 and 328 kN for S2-1-450. Thus, the shear capacity of the strengthened PHC slabs relies on both of the concrete and the CFRP properties.

The load-deflection curves between S1-2-450 and S2-2-450 are shown in Figure
3.22(c). Both curves exhibited similar linear behavior within the elastic range. The load-deflection curves of both specimens also showed similar improvement on ductility. However, the ultimate load of the medium-prestressing specimen showed higher improvement than that of low-prestressing. This could be caused by the material properties of the S1-series specimens as mentioned earlier, i.e. bonding effect between prestressing strands and concrete, the occurrence of vertical flexure crack before failure and pre-existing fine cracks in the concrete, which could reduce the shear strengthening performance of PHC slabs.

In conclusion, based on the above discussion, the prestressing level would not necessarily affect the shear capacity of non-strengthened and strengthened PHC slabs.

(a) Control specimen
Figure 3.22 Effect of prestressing level on the load-deflection relation of S1-seris and S2-series specimens
3.8 Summary of experimental study

1. According to the experimental results, using externally bonded CFRP sheets along the internal perimeter axis of the slab voids with the fiber orientation perpendicular to the member longitudinal axis can considerably improve the shear capacity of full-width PHC slabs.

2. CFRP sheets play similar role as shear stirrups. Thus, increasing layers of CFRP sheets would considerably enhance the shear performance of PHC slabs. The experimental results showed that the full width PHC slabs strengthened by 1 layer CFRP sheet had an improvement of shear capacity from 17% to 22%. However, the specimens strengthened by 2 layers CFRP sheets showed more enhancement of shear capacity from 11% to 38%.

3. It is more efficient to cover the critical shear-tension region when applying the longer CFRP sheets. The PHC slabs strengthened by 450 mm long CFRP sheets showed more improvement (27%) on shear strengthening effect than that of 300 mm long CFRP sheets.

4. Higher prestressing level may not necessarily help to enhance the shear capacity of PHC slabs. The shear capacity of strengthened PHC slabs is mainly related to the material properties of concrete and CFRP sheets.

5. All specimens failed in typical shear-tension failure and no debonding failure occurred between the CFRP sheets and concrete surface before failure.
6. Based on test results, 2 layers of CFRP sheets bonded over 450 mm are recommended in order to achieve considerable increase in the shear capacity.
CHAPTER 4 FINITE ELEMENT SIMULATION

Finite element analysis (FEA) is an effective and economic method to study the complex behavior of any construction members, especially for those having complicated non-linear structural behavior. ABAQUS is one of the most commonly used commercial finite element analysis software for understanding the structural mechanism and conducting parametric studies. In the current research, ABAQUS 6.14 will be used to numerically simulate the behavior of CFRP-strengthened PHC slabs.

Although experimental test is the most reliable way to study the behavior of structures, it has several disadvantages. The main issue is that the experimental results could be different in each test (as in the case of three S2-2-450 specimens) which could be caused by various reasons. Besides, experimental test is limited, expensive and time-consuming. Thus, FE simulation is an important way not only to cross check the experimental results, but also perform parametric study.

In this chapter, the details of the developed FE model will be presented first, which includes the element types, the constitutive model, the mesh size and the boundary conditions of each component. Then, the validity of the FE model will be verified by comparing the numerical simulation results with the experimental ones.

4.1 Constitutive Relationship

4.1.1 Concrete

Concrete is a composite material composed of coarse aggregate, fine aggregate,
cement and water which hardens over time. The inhomogeneous material constitutive property makes concrete a typical type of inhomogeneous structural member with complicated failure mechanism. An accurate concrete model depends on many parameters and functions obtained from experimental study. Therefore, a proper material model in FEA should be able to simulate both elastic and plastic behavior of concrete in compression and tension (Wahalathantri, 2011).

There are two concrete constitutive models in ABAQUS: concrete smeared cracking model and concrete damage plasticity model. The smeared crack concrete model is mainly used to simulate the formation of crack onsets at any location when the concrete stress reaches one of the failure surfaces either in the biaxial tension region or in a combined tension-compressive region. The concrete smeared crack model provides a general behavior of modeled concrete in all structural types, i.e. beams, trusses, shells and solids.

In this study, the concrete damage plasticity (CDP) model is used to simulate the concrete behavior of PHC slabs. According to ABAQUS Theory Manual 6.14 (2014), the CDP model aims at analyzing the general behavior of concrete structures subjected to cyclic and/or dynamic loads. Concrete behaves in a brittle manner (cracking in tension and crushing in compression) under low confining pressures. The studied materials should satisfy the following macroscopic properties: different yield strength in tension and compression, the softening behavior after yielding in tension and the
softening-hardening behavior after yielding in compression, different reduction factor of the elastic stiffness, stiffness recovery during cyclic loading.

In the CPD model, the main failure mechanism of concrete is assumed to be caused by cracking in tension and crushing in compression. The yield or failure is controlled by two variables: the equivalent tensile plastic strain and the equivalent compressive plastic strain. The simulated response of concrete to uniaxial loading in tension and compression by the CPD model is shown in Figure 4.1.

(a) Stain-stress response due to tension
Figure 4.1 (a) shows the stress-strain relationship of concrete under uniaxial tension. The stress-strain behavior of concrete follows a linear elastic relation before reaching the ultimate failure stress $\sigma_{t0}$. Micro-cracking initiates in the concrete at the failure stress. After the failure stress, a softening stress-strain response represents the formation of micro-cracks and the increasing of the strain in concrete. The stress-strain relationship of concrete under uniaxial compression also shows a linear trend before reaching the yielding stress $\sigma_{c0}$, as shown in Figure 4.1(b). The plastic range consists by two parts, hardening stress-strain response before reaching the ultimate stress $\sigma_{cu}$ and a strain softening response beyond the ultimate stress $\sigma_{cu}$.

In this model, the unloading response is weakened and the elastic stiffness of the material appears to be degraded when concrete specimen is unloaded from any point on
the strain softening part of the stress-strain relationship in Figure 4.1. The degraded response of concrete is featured by two independent variables, \( d_t \) and \( d_c \). If \( E_0 \) is the initial elastic stiffness of the concrete, the stress-strain responses under uniaxial tension and compression forces are, respectively:

\[
\sigma_t = (1 - d_t)E_0\left(\varepsilon_t - \tilde{\varepsilon}_t^{pl}\right) \tag{4.1}
\]

\[
\sigma_c = (1 - d_c)E_0\left(\varepsilon_c - \tilde{\varepsilon}_c^{pl}\right) \tag{4.2}
\]

The effective uniaxial cohesion tensile and compressive stresses are given by Eq. (4.3) and (4.4). The effective cohesion stresses determine the size of the yield surface.

\[
\bar{\sigma}_t = \frac{\sigma_t}{(1 - d_t)} = E_0\left(\varepsilon_t - \tilde{\varepsilon}_t^{pl}\right) \tag{4.3}
\]

\[
\bar{\sigma}_c = \frac{\sigma_c}{(1 - d_c)} = E_0\left(\varepsilon_c - \tilde{\varepsilon}_c^{pl}\right) \tag{4.4}
\]

where, \( \tilde{\varepsilon}_t^{pl} \) and \( \tilde{\varepsilon}_c^{pl} \) are hardening variables, which are referred to equivalent plastic strain in tension and compression, respectively.

### 4.1.2 Reinforcement

The main function of steel strands in PHC slabs is to resist the formation of cracks in concrete to enhance the tensile resistance and ductility of the member (Celal, 2011). In this case, the prestressing strands in PHC slabs belong to high strength steel which does not have actual yield stage. Thus, the constitutive relation curve of the reinforcement should include the elastic stage, yielding stage and the hardening stage. The transition zone between the elastic range and the plastic range was connected by a smooth curve. The constitutive model of the reinforcement in this study was based on the dual
slash-curve model (Figure 4.2) reported by Yu (2002). The stress-strain relationship of the reinforcement can be represented as

\[
\sigma_s = \frac{\sigma_b \varepsilon_b - \sigma_a \varepsilon_a}{\varepsilon_b - \varepsilon_a} + \frac{\varepsilon_a \varepsilon_b (\sigma_a - \sigma_b)}{\varepsilon_s (\varepsilon_b - \varepsilon_a)}
\]  \((4.5)\)

where, \(\sigma_a, \sigma_b, \varepsilon_a\) and \(\varepsilon_b\) are the stress and strain at points \(a\) and \(b\), respectively. The range between \(a\) and \(b\) is the softening part of the steel.

![Stress-strain curve for reinforcement](image)

**Figure 4.2 Stress-strain curve for reinforcement**

### 4.1.3 FRP

Based on the reviewed literature, the behavior of FRP composite applied to shear strengthening of PHC slab could be assumed as linear elastic (Coronado et al., 2006, Ebead et al., 2002, El Sayed et al, 2005 and Chen et al, 2012). The constitutive model applied in finite element analysis is shown in Figure 4.3.
4.2 Element Type and Material Properties

4.2.1 Concrete

The element type used to model concrete was C3D8R in ABAQUS. Figure 4.4 shows this three-dimensional, 8-node, reduced-integration solid element which can be used for linear analysis as well as complex nonlinear analysis involving contact, plasticity, and large deformations. Besides, this element is able to simulate plastic deformation, crushing, cracking and even shear transfer at crack interface without shear locking problem.

A total of thirty 100 × 200 mm (4 × 8 in) and fifteen 150 × 300 mm (6 × 12 in)
cylinders were prepared and cured under the same conditions of PHC slab specimens to
determine the compressive and tensile strength of concrete. The average result of the
concrete ultimate compressive strength $f'_{c}$ obtained from the compression test are 59.3
MPa for S1 specimens (low-prestressing PHC slabs with 6 prestressing strands) and 52.2
MPa for S2 specimens (medium-prestressing PHC slabs with 8 prestrssing strands). The
Young’s modulus $E_{c}$ is taken as 22167 MPa for the S1 specimens and 23946 MPa for the
S2 specimens, respectively. The Poisson’s ratio of concrete is assumed to be 0.26 in the
FE model. The average tensile strength of concrete is taken as 3.35 MPa which is
determined by standard splitting test on fifteen concrete cylinders. The other parameters
of concrete required by the CDP model in ABAQUS are listed in Table 4.1.

<table>
<thead>
<tr>
<th>Dilation Angle $\psi$</th>
<th>Eccentricity</th>
<th>$f_{biol}/f_{c0}$</th>
<th>$K_c$</th>
<th>Viscosity Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>0.1</td>
<td>1.16</td>
<td>0.667</td>
<td>0.0005</td>
</tr>
</tbody>
</table>

As mentioned in the ABAQUS Theory Manual 6.14 (2014) and the study of Kmiecik
et al. (2011), the definitions of the above parameters are listed below:

(a) The dilation angle, $\psi$ describes the performance of concrete under compound
stress. The value of the dilation angle could be taken as concrete internal friction
angle (30°-40° for concrete).

(b) The eccentricity can be calculated as a ratio between the tensile strength and the
compressive strength which is recommended to be assumed as 0.1.

(c) $f_{biol}/f_{c0}$ is a ratio between the strength in the biaxial state and that in the uniaxial
state. The ABAQUS user’s manual specifies the default value to be 1.16.

(d) $K_c$ is a ratio of the distances between the hydrostatic axis and respectively the compression meridian and the tension meridian in the deviatoric cross section. The value of 2/3 is typical for concrete.

(e) The viscosity parameter indicates the relaxation time of the visco-plastic system. It is used for the visco-plastic regularization of the concrete constitutive equations in ABAQUS (Standard) analysis. In ABAQUS, the default value of this parameter is 0.0. However, to help achieve convergence during analysis, it is usually set as $\mu = 0.0005$.

### 4.2.2 Reinforcement

The element type T3D2 (two-node, three-dimensional truss element) was used to model the prestressing strands of PHC slabs. Truss element can be used to simulate the long, slender structural members that can transmit only axial compressive and tensile force but not moment. The geometry of the two-node truss element is shown in Figure 4.5, which is defined by two nodes at each end to form a straight line. On the other hand, the characteristic of the three dimensions was set by the cross-sectional area of the strands.
The properties of the prestressing strands were provided by the supplier (PSI Inc.), which are shown in Table 4.2.

<table>
<thead>
<tr>
<th>ID</th>
<th>7-wire, low-relaxation strands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>13 mm</td>
</tr>
<tr>
<td>Ultimate tensile strength</td>
<td>1860 MPa (270 ksi)</td>
</tr>
<tr>
<td>Modulus of elasticity ($E_p$)</td>
<td>196,500 MPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The prestressing strands do not have recognizable yielding like other regular mild steel and no strain hardening exists before rupture. Based on PCI manual for design of hollow core slabs (1998), a stress-strain relation for low relaxation strands of Grade 270ksi (1860 MPa) was given in Eq. (4.13) and Eq. (4.14), and portrayed in Figure 4.6.
Figure 4.6 Stress-strain curve or strand elements

\[ \varepsilon_{ps} \leq 0.0086: \quad f_{ps} = 28500 \times \varepsilon_{ps} \text{ (ksi)} \quad (4.6) \]

\[ \varepsilon_{ps} > 0.0086: \quad f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \text{ (ksi)} \quad (4.7) \]

In addition, the prestress loss due to elastic shortening (ES) of concrete should be considered in this study, which is introduced by Eq. (4.15)

\[ ES = \frac{K_{es}E_{ps}f_{ci}}{E_{ci}} \quad (4.8) \]

where \( K_{es} \) is the factor for calculating prestress loss due to elastic shortening, which can be taken as 1.0 for pretensioned members, \( E_{ps} \) is the modulus of elasticity of prestressing tendons, \( E_{ci} \) is the modulus of elasticity of concrete at time of introducing initial prestress,
$f_{cir}$ is the net compressive stress in concrete at the centroid of prestressed reinforcement at the time of applying initial prestress.

$$f_{cir} = K_{cir} \left( \frac{P_i}{A_g} + \frac{P_i e^2}{I_g} \right) - \frac{M_g e}{I_g} \quad (4.9)$$

where $K_{cir}$ is the factor for calculating elastic shortening prestress losses, which is taken as 0.9 for pretensioned members, $P_i$ is the initial prestressing force after seating loss, $e$ is the distance from the neutral axis to the centroid of the prestressed reinforcement, $A_g$ is the area of the gross concrete section at the cross section considered, $I_g$ is the moment of inertia of gross concrete section at the cross section considered, $M_g$ is the unfactored self-weight moment.

In ABAQUS, the prestressing force in strand is simulated by a specific temperature load to make the strands to shrink which could apply the prestressing force in concrete. The equation is shown below:

$$T = \frac{-F}{E_p \times A \times e_c} \quad (4.10)$$

where $T$ is the loading temperature, $F$ is the required prestressing force, $E_p$ is the elastic modulus of reinforcement, $A$ is the cross-sectional area of the strand (the effective cross-sectional area of the strand is 100 mm$^2$ in the current study), and $e_c$ is the thermal expansion coefficient of the strand (the used value is 0.00001).

### 4.2.3 FRP

Shell element is used to model structures which have one leading dimension and the thickness is significantly smaller than the other dimensions. It is suitable to simulate the
CFRP sheets that glued onto the PHC slab voids. In ABAQUS, there are two kinds of shell element, the conventional shell element and the continuum shell element, as shown in Figure 4.7. The conventional shell element is used to discretize a body by defining the geometry at a reference surface, the thickness of which is defined through the section property definition. This type of shell element has both displacement and rotational degrees-of-freedom. On the contrary, the continuum shell element discretizes an entire three-dimensional body which looks like three-dimensional continuum solids. The element has only displacement degree-of-freedom, and its thickness is determined from the element nodal geometry. However, the conventional shell element and continuum shell element have similar kinematic and constitutive behavior. The element type used to model CFRP sheets in this study the four-node quadrilateral in-plane stress/displacement shell element with reduced integration and a large-strain formulation (S4R) (AB AQUS User’s Manual 6.14, 2014). The direction of the simulated CFRP sheets was perpendicular to the longitudinal axis of the slab. Based on the reviewed literature and experimental observations, the interface between the concrete and the CFRP sheets is simulated by tie constraint.
The strengthening materials, including CFRP sheets and epoxy, were provided by Sika Canada Inc. Sika Wrap-900C CFRP sheets and Sikadur300 epoxy were used to strengthen the PHC slabs specimens. The SikaWrap-900C is a uni-directional, fleece stabilized, stitched and heavy carbon fiber fabric for the wet application process of structural strengthening (SIKA Canada). The mechanical properties of the CFRP sheets with epoxy resin are either provided by the supplier or determined by tensile tests on representative samples in accordance with ASTM D3039 (ASTM D3039/D3039M, 2014). The results showed a linear-elastic stress-strain relationship with average values of elastic modulus and tensile strength being 94000 MPa and 1012 MPa, respectively.

### 4.2.4 Steel bearing plate

The bearing plates are simulated by the eight-node solid element (C3D8R) which is the same as the element type selected to model concrete. The material properties of the
bearing plates are assumed to be linear-elastic which contains two essential parameters, the modulus of elasticity (200GPa) and the Poisson’s ratio (0.3). The simulated bearing plates do not crush or crack and could help to simulate the boundary condition of the slab.

4.2.5 Reinforcement-concrete interface

The interface between the reinforcement and the concrete was simulated by the embedded element. The embedded element technique can be used to simulate reinforcement, shell or surface element that lie embedded in a set of solid element. The embedded technique will not restrict rotational degree-of-freedom when shell or beam elements are embedded in solid element (ABAQUS User’s Manual 6.14, 2014). In the current study, the steel strands lie embedded into the host element concrete. Thus, the translational degrees-of–freedom at the node of the embedded element are constrained by the corresponding degrees-of-freedom of the host element.

4.3 Geometry and boundary conditions

The cross-section of the low and medium prestressing PHC slabs is shown in Figures 4.8 and 4.9, respectively. Figures 4.10 (a) and 4.10 (b) give a 3D view of the geometry of the FE model developed for the control slab and the strengthened slab. The X-axis is along the longitudinal direction of the slab, the Y-Z plane represents the cross section of the slab.
Figure 4.8 Cross-section of low prestressing HPC slab

Figure 4.9 Cross-section of medium prestressing HPC slab

(a) Control slab
In the experimental study, all slabs were tested under simply supported condition. The support near the loading point is a pin and another is a roller. To simulate the pin support, all nodes on the bottom of the bearing plate are constrained in the x, y, and z directions (Figure 4.1). Besides, the nodes on the bottom surface of the bearing plate on the other end are constrained only in the y and z directions (Figure 4.1). The displacement of the constrained nodes is set as zero to satisfy the actual test condition.

4.4 Solution Control

The finite element analysis is divided into two steps. The first step should simulate the application of the prestress provided by prestressing strands. This is controlled by
decreasing temperature of the strands as mentioned earlier to make the concrete shrink. “*Model Change” command is necessary to be used in the first step to remove the tie constraints between the steel bearing plates and the concrete, as well as those between the CFRP sheets and the concrete. In the second step, the PHC slab is simulated to be subjected to a gradually applied line load, as in the physical tests. The load increment is taken as 1.0 kN and the magnitude of the load increases until the failure of the slab.

In this FE model, the nonlinear equilibrium equations are solved by the Full Newton method. The convergence criterion is based on force. The tolerance limits of the first step and the second step are 0.05 and 0.005, respectively. The numerical simulation process will be ended when the solution is hard to converge.

4.5 Validation of Finite Element Model

The load-deflection relationship reflects the behavior of the studied members during the entire loading history. Therefore, it is an important indication of the validity of the FE model. Figure 4.12 gives the comparison of the load-deflection relationship of each specimen between the FE model prediction and the experimental data. It can be seen from the subplots in Figure 4.12 that the developed finite element model can satisfactorily predict behavior of the non-strengthened and strengthened PHC slabs in terms of the initial stiffness, the ultimate failure load and the deflection corresponding to failure.

However, the experimental result of the S1-1-450 specimen obtained from the experimental study showed a bi-linear range before failure. This could be caused by the
pre-existing cracks in the PHC slab which cannot be simulated in the FE modelling. Although the load-deflection curve obtained from FEM of S1-1-450 could not well match that from the experiment data, the predicted ultimate load was still close to the experimental result. Both load-deflection curves of S1-2-450 obtained from experiment and FE simulation exhibited similar behavior within the elastic range, whereas the experimental ultimate load of the S1-2-450 specimen showed 18% difference compared to the data predicted by FE model. This is believed to be caused by the inhomogeneous property of concrete and the pre-existing fine cracks in the PHC slabs which cannot be simulated in ABAQUS.
Figure 4.12 Load-deflection relationships

(f) S2-2-300

(g) S2-2-450

0 5 10 15
0 60 120 180 240 300 360 420
Load (kN)
Displacement at loading position (mm)

Experiment
FEM

EXP-S2-2-450
EXP-S2-2-450-2nd
EXP-S2-2-450-3rd
FEM
Figure 4.13 shows the load-deflection behavior curves of S1-series predicted by finite element simulation. The ultimate loads obtained from experimental study and finite element simulation are summarized in Table 4.3. The differences of control specimen and S1-1-450 are under 5.1%. However, S1-2-450 showed much larger difference of 18.4%. The reason of this inconsistence result will be explained later. In Figure 4.12 All curves exhibited a similar linear trend within the elastic range. Besides, the shear capacity of strengthened specimens improved considerably, i.e. 23% for S1-1-450 and 29% for S1-2-450. Furthermore, the ultimate loads obtained from numerical simulation were always higher than the experimental ones. This could be due to the presence of imperfections in the PHC slabs such as fine cracks, uneven distribution of coarse and fine aggregates in real specimens, which would degrade the slab performance. These factors are not included in the numerical simulation.
Figure 4.13 Load-deflection behavior of S1-series obtained from FEM

Table 4.3 Comparison of S1-Series results

<table>
<thead>
<tr>
<th>FE Model-ID</th>
<th>Failure load (kN)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experiment</td>
<td>ABAQUS</td>
</tr>
<tr>
<td>S1-C</td>
<td>291</td>
<td>304</td>
</tr>
<tr>
<td>S1-1-450</td>
<td>356</td>
<td>374</td>
</tr>
<tr>
<td>S1-2-450-series</td>
<td>332</td>
<td>393</td>
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</table>

Figure 4.14 shows the finite element simulation results of the load-deflection relationship of the S2-series PHC slab specimens. Both the experimental and numerical results are summarized in Table 4.4. The failure load predicted by the finite element model compared well with the experimental results. All specimens also showed similar behavior during the elastic range. The maximum error between the experimental results and numerical prediction is less than 8% except the S2-2-450-2nd specimen has a difference of 10.7%. This is believed to be caused by the local debonding of CFRP sheets before the test, as mentioned in Chapter 3.
Figure 4.14 Load-deflection behavior of S2-series obtained from FEM
Table 4.4 Comparison of S2-Series results

<table>
<thead>
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<th>FE Model-ID</th>
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<tr>
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<td>Experiment</td>
<td>ABAQUS</td>
<td>Difference</td>
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<tr>
<td>S2-C</td>
<td>281</td>
<td>298</td>
<td>6.2%</td>
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<tr>
<td>S2-1-450</td>
<td>328</td>
<td>342</td>
<td>4.3%</td>
</tr>
<tr>
<td>S2-2-300</td>
<td>310</td>
<td>333</td>
<td>7.4%</td>
</tr>
<tr>
<td>S2-2-450</td>
<td>387</td>
<td>405</td>
<td>4.7%</td>
</tr>
<tr>
<td>S2-2-450-2nd</td>
<td>368</td>
<td>405</td>
<td>10.5%</td>
</tr>
<tr>
<td>S2-2-450-3rd</td>
<td>378</td>
<td>405</td>
<td>7.1%</td>
</tr>
</tbody>
</table>

The effect of prestressing level on the non-strengthened and strengthened PHC slabs obtained from numerical simulation is shown in Table 4.5. Results show that the ultimate load of the non-strengthened and strengthened slabs obtained from experiments and numerical simulation agree well. The difference of the ultimate load between the experimental results and the ABAQUS results of non-strengthened specimens are 4.5% for low-prestressing specimens and 4.9% for medium-prestressing specimens, respectively. Similarly, the maximum difference between the low-prestressing and medium-prestressing specimen strengthened by 1 layer 450 mm long CFRP sheets is only 5.1%. On the other hand, the predicted ultimate loads of S1-series are always higher than the S2-series specimens under the same strengthening condition, which is consistent with the experimental results. Thus, the developed finite element model is capable to simulate the shear behavior of the non-strengthened and strengthened PHC slabs under two different prestressing levels.
### Table 4.5 Effect of prestressing level on S1-series and S2-series

<table>
<thead>
<tr>
<th>FE Model-ID</th>
<th>S1-C</th>
<th>S2-C</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Failure load (kN)</strong></td>
<td>Experiment</td>
<td>291</td>
</tr>
<tr>
<td></td>
<td>ABAQUS</td>
<td>304</td>
</tr>
<tr>
<td><strong>Difference</strong></td>
<td>4.5%</td>
<td>4.9%</td>
</tr>
<tr>
<td><strong>Failure load (kN)</strong></td>
<td>S1-1-450</td>
<td>356</td>
</tr>
<tr>
<td></td>
<td>ABAQUS</td>
<td>374</td>
</tr>
<tr>
<td><strong>Difference</strong></td>
<td>5.1%</td>
<td>4.3%</td>
</tr>
</tbody>
</table>

Additionally, the relationships between the load and the top longitudinal concrete compressive strain are shown in Figure 4.15. The difference of the compressive strain corresponding to the failure load between the numerical prediction and the experimental results varies from 3.8% to 12.1%. Besides, some sample relationships between the load and the longitudinal tensile strain in CFRP sheets are portrayed in Figure 4.16. The maximum difference of the tensile strain corresponding to the failure load is 12.9%. Thus, the developed FE model is considered to be capable of predicting the compression in the concrete and the tension in the CFRP sheets.
(a) S1-C (max. difference: 7%)

(b) S2-C (max. difference: 3.8%)
Figure 4.15 Load vs top longitudinal concrete compressive strain relationships

(c) S1-1-450 (max. difference: 2.2%)

(d) S2-2-450 (max. difference: 12.1%)

Figure 4.15 Load vs top longitudinal concrete compressive strain relationships
Figure 4.16 Load vs top longitudinal tensile strain in CFRP sheets (middle web)
Figure 4.17 gives some sample comparison between the actual cracking pattern and the plastic strain (PEEQ) results obtained by numerical simulation. Results show that the plastic strain and crack growth trend within the shear-tension region predicted by the FE model are consistent with the formed crack in the experiments.

(a) S1-C

(b) S1-1-450
4.6 Parametric Study

Based on the above validations, the FE model developed in the current research is capable of properly simulating the behavior of the PHC slabs with and without strengthening by CFRP. Therefore, it can be used to conduct parametric study. Two important parameters were investigated in the experimental study, i.e. the strengthening length and the thickness of the CFRP sheets. Besides, during the experiments, in order to apply CFRP sheet easily, the CFRP sheets were cut into two pieces and arranged on each side of the slab internal void (Figure 3.2), covering an arc range of 150° on each side.

Based on the results obtained from Phase I, for PHC web beams, the specimens strengthened by the CFRP sheets with a length of 600 mm showed more improvement of shear capacity than the other cases. However, it is challenging to install 600 mm CFRP sheets in the PHC slab voids on site. Therefore, the parametric study conducted in this
section will focus on the impact of CFRP strengthening width (i.e. the arc range of CFRP sheets) and strengthening thickness (i.e. the number of layers of CFRP sheets) in affecting the strengthening effectiveness based on a strengthening length of 300 mm and 450 mm. The same geometric details and material properties as those in the earlier sections were used in the parametric study.

4.6.1 Strengthening thickness

Three different thicknesses of CFRP sheets, i.e. 1 layer, 2 layers and 3 layers in each void of PHC slabs, were applied to evaluate the shear strengthening effectiveness. Figure 4.18 gives sample load-deflection relationships of S2-series-300 and S2-series-450 specimens strengthened by three different thicknesses of CFRP sheets.

The ultimate load and enhancing rate of each case obtained from FE model are portrayed in Figure 4.18 and summarized in Table 4.6. For the S2-series-300 model, the ultimate load of 1-, 2- and 3-layers models are 313kN, 333kN and 348kN, respectively. For a strengthening length of 300 mm, the average enhancement of shear capacity by each layer is 5.6%. The S2-series-450 models show a similar growth trend. The model strengthened by 3 layers CFRP sheets shows a significant improvement of shear capacity by 53.4% with the ultimate load reaches 457kN. The ultimate load of the S2-1-450 model and the S2-2-450 model obtained from finite element simulation are respectively 342kN and 405kN. With a CFRP strengthening length of 450 mm, the average strengthening effect provided by each layer of CFRP sheet is 19.3%.
Figure 4.18 FEM parametric study results of strengthening thickness effect
Table 4.6 Comparison between strengthening thickness effect by FE model

<table>
<thead>
<tr>
<th>FE Model-ID</th>
<th>Strengthening thickness (layers)</th>
<th>Failure load (kN)</th>
<th>Enhancing rate compared to S2-C (298kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-series-300</td>
<td>1</td>
<td>313</td>
<td>5.0%</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>333</td>
<td>11.7%</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>348</td>
<td>16.1%</td>
</tr>
<tr>
<td>S2-series-450</td>
<td>1</td>
<td>342</td>
<td>14.8%</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>405</td>
<td>35.9%</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>457</td>
<td>53.4%</td>
</tr>
</tbody>
</table>

4.6.2 Strengthening width

The shear force in PHC slabs is mainly resisted by the concrete in the webs. In the current research, the CFRP sheets are installed along the internal perimeter of the slab voids. The enhanced shear capacity of the strengthened PHC slabs are provided by the tensile strength of the CFRP sheets. Since the maximum shear stress occurs at the mid-height of the web, it is reasonable to expect that only part of the CFRP sheets, i.e. those close to the mid-height of the web, would be the main contributor to the shear capacity enhancement. Thus, it is worth to study the effective strengthening width in three scenarios, i.e. with an arc range of 150°, 120° and 90°, respectively, which are shown in Figure 4.19. Besides, reducing the strengthening width of CFRP sheets would also bring down the cost and could help to make this technique more economic.
Table 4.7 shows the FEM results of four different CFRP strengthening widths based on the S2-2-450 FE model. It is interesting to note that the 120° case shows a very close ultimate load to the 150° case. Theoretically, this result indicates that a width of CFRP sheets corresponding to a 120° arc range could adequately cover the full critical shear tension region. With the width of CFRP sheets decreases, the ultimate shear capacity of PHC slab drops gradually. Even a strengthening width corresponding to a 90° arc range still shows a considerable enhancement of shear capacity by 32% compared to the
non-strengthened case.

Table 4.7 S2-2-450 FEM parametric study results

<table>
<thead>
<tr>
<th>Strengthening width</th>
<th>Failure load (kN)</th>
<th>Percentage of reduction (compare with 150°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>150°</td>
<td>405</td>
<td>-</td>
</tr>
<tr>
<td>120°</td>
<td>402</td>
<td>-</td>
</tr>
<tr>
<td>90°</td>
<td>389</td>
<td>3.9%</td>
</tr>
</tbody>
</table>

Additionally, the principle tensile stress results in CFRP sheets are given in Figure 4.20. Obviously, the critical tensile region in each case is located in the middle portion of the CFRP sheets and it expands to both sides along the vertical direction of CFRP fiber. According to Figure 4.20(c), the critical shear-tension region is still within the strengthened zone which helps to prove that the 90° strengthening width of CFRP sheets could enhance the shear capacity efficiently.

![Diagram](image)
4.7 Summary of numerical simulation

1. The finite element model developed in ABAQUS is capable of simulating the shear behavior of the non-strengthened and the strengthened PHC slab specimens in terms of load-deflection relationships, load-strain relationships and principle tensile stress in CFRP sheets.

2. It is verified by the finite element simulation that the prestressing level may not necessarily affect the shear capacity of non-strengthened and strengthened PHC slabs. The shear resistance of PHC slabs is mainly influenced by the concrete property of PHC slabs and the applied CFRP sheets.

3. Parametric study results show that with a certain length of strengthening zone, reducing the width of the applied CFRP sheets from 150° arc range to 90° arc range would slightly degrade the shear strengthening performance of this technique. Furthermore, increasing the thickness of the applied CFRP sheets can considerably enhance the shear performance of PHC slabs, especially when the strengthening zone is longer.
CHAPTER 5 CONCLUSION AND RECOMANDATION

5.1 Conclusion

A new shear strengthening technique for PHC slabs by installing externally bonded CFRP sheets to each void of the full width PHC slab has been proposed. In order to evaluate the feasibility and effectiveness of this new method, ten full width PHC slab specimens strengthened by CFRP sheets externally bonded along the circumferential direction of the surface of PHC slab voids were tested. The studied parameters include the prestressing level, the length and the thickness of the strengthening zone. The experimental results showed a considerable enhancement on shear capacity of the strengthened PHC slabs. In addition, some strengthened specimens also showed sizable improvement on the ductility. Thus, the proposed technique is effective on the shear strengthening of PHC slabs.

In the numerical study, a finite element simulation in ABAQUS was conducted to investigate the shear behavior of full width PHC slabs strengthened by externally bonded CFRP sheets. The developed finite element model was validated by the experimental results and a parametric study was also carried out to evaluate the strengthening effect under different strengthening parameters. The developed finite element model by ABAQUS could reasonably well simulate the shear behavior of CFRP-strengthened full width PHC slab. The load-deflection and load-strain relationships predicted by numerical simulation could match well with the experimental data.
The main findings obtained from the experimental study and numerical simulation can be summarized as follow:

1. Effect of strengthening length:
   The results obtained from the experimental study indicated that a longer CFRP strengthening length can provide more shear resistance since it can cover more of the critical shear-tension region. The shear capacity enhancement of the strengthened PHC slabs are 11% for the S2-2-300 specimen and 34% for the S2-2-450-series, respectively. These results were also validated by finite element simulation.

2. Effect of strengthening thickness:
   Both experimental study and numerical simulation showed that increasing the strengthening thickness by applying more layers of CFRP sheets could provide more improvement to shear capacity of PHC slabs. The role of the CFRP sheets can be considered as shear stirrup. Furthermore, numerical simulation results showed that the average contribution of each layer of the applied CFRP sheet was about 20%.

3. Effect of prestressing level:
   The current tests included two prestressing levels of PHC slabs, i.e. the low-prestressing level (6 prestressing strands) and the medium-prestressing level (8 prestressing strands). Results obtained from experimental study and numerical
simulation both indicated that higher prestressing level might not necessarily help to enhance the shear capacity of PHC slabs. The shear capacity of non-strengthened and strengthened PHC slabs is mainly related to the concrete property of PHC slab itself and the applied CFRP material.

4. Effect of strengthening width:

The effect of strengthening width on the proposed shear strengthening technique was only investigated in the numerical simulation. Results showed that with a certain strengthening length and thickness, reducing the width of the applied CFRP sheets could slightly degrade the shear strengthening performance of PHC slabs. In addition, it was found that an arc range of 90° is sufficient to cover the critical shear portion of each web in PHC slab and showed 32% shear resistance improvement.

5.2 Future work

According to the results and conclusion in the current study, the recommendations for future study are listed below:

1. The current study only investigated three parameters, i.e. the prestressing level of PHC slab, the thickness of the strengthening zone and the length of the strengthening zone. There are still a number of other parameters that should be evaluated in the future study, for example, the type of FRP material and the type of epoxy.
2. According to the results from parametric study of finite element simulation, the effectiveness of strengthening width should be further studied in the future tests.
APPENDIX A:

Load vs longitudinal tensile strain in CFRP sheets relationships obtained from the first and the second strain-gauges
Load vs longitudinal tensile strain in CFRP sheets relationships obtained from the first strain gauge

(a) S2-2-300 and S2-2-450

(b) S2-1-450 and S2-2-450-series
Load vs longitudinal tensile strain in CFRP sheets relationships obtained from the second strain gauge
Load vs longitudinal tensile strain in CFRP sheets relationships obtained from the third strain gauge

(b) S2-1-450 and S2-2-450-series
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<table>
<thead>
<tr>
<th>NAME:</th>
<th>Xianzhe Meng</th>
</tr>
</thead>
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<tr>
<td>PLACE OF BIRTH:</td>
<td>Beijing, China</td>
</tr>
<tr>
<td>YEAR OF BIRTH:</td>
<td>1991</td>
</tr>
<tr>
<td>EDUCATION:</td>
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</tr>
<tr>
<td></td>
<td>2009-2013 Bachelor degree in Civil Engineering</td>
</tr>
<tr>
<td></td>
<td>University of Windsor, Windsor, ON, Canada</td>
</tr>
<tr>
<td></td>
<td>2014-2016 M.A.Sc. Civil Engineering/Structural</td>
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