

An ACI Technical Publication

A SYMPOSIUM HONORING KHALED SOUDKI



1967-2013

Towards
Sustainable
Infrastructure
with Fiber
Reinforced
Polymer
Composites

Editor:
Raafat El-Hacha

SP-322



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In Tribute to Professor Khaled Soudki

Professor Khaled Soudki, PhD, PEng, FACI, passed away on September 17, 2013 after a long battle with cancer. He is survived by his wife, Rana Shami, and daughter Sara and son Adnan. He was a graduate of the American University of Beirut (BEng, 1987), Cornell University (MSc, 1989) and the University of Manitoba (PhD, 1994), and held a postdoctoral position at Queens University in Kingston, Canada. Professor Soudki joined the Department of Civil and Environmental Engineering at the University of Waterloo as an Assistant Professor in 1996. A prolific and well-respected researcher, he quickly progressed to Associate Professor in 2001 and full Professor in 2006. He was appointed Canada Research Chair in Innovative Structural Rehabilitation in 2003 and was reappointed in 2006 for a further five-year term. He was also the founding Dean of Engineering at the American University in Dubai, UAE and a visiting scholar at the Swiss Federal Laboratory for Material Testing (Empa) in Dübendorf, Zurich, Switzerland. He was very active on many international scientific committees and was a Fellow of the American Concrete Institute. Khaled was an internationally distinguished researcher in the field of reinforced and prestressed concrete structures with emphasis on the use of advanced fibre reinforced polymers for the repair of structures. His research on the repair of corrosion-damaged reinforced concrete structures led the field and significantly advanced the application of fiber-reinforced polymers. His work in prestressing of fiber-reinforced polymers was similarly groundbreaking, including the development of an anchorage system that is now widely used. Khaled leaves behind a legacy of scientific research in Canada and around the world. His research contributions resulted in more than 300 publications, including patents, book chapters and many journal and conference publications. His publications have been cited more than 1000 times. He supervised 20 PhD students, 25 Master's students and 10 postdoctoral fellows; 15 former students and postdocs have gone on to careers in academia in universities around the world.

On a personal note, I mourn the loss of a true friend and brother. I will always remember Khaled as an outstanding researcher, talented professor, knowledgeable scholar, successful colleague, mentor, kind and humble person, and a true gentleman. His passing is a great loss to the field and to all of us as his friends. Words are not enough to express our sorrow for losing him. I am proud to have had the opportunity to be his co-author on the invited paper "Prestressed near-surface mounted fibre reinforced polymer reinforcement for concrete structures — a review". This paper is one of a selection of papers in the special issue of the Canadian Journal of Civil Engineering to honour the 20th anniversary of the Advanced Composite Materials in Bridges and Structures (ACMBS) conference Series. His last email to me on August 29, 2013 when he was in a critical condition at the hospital: "Congrats on our joint publication". Unfortunately, he was not with us on May 30, 2014 in Halifax at the Canadian Society for Civil Engineering (CSCE) Awards Gala Dinner when this paper won the Casimir Gzowski Gold Medal for the Best Civil Engineering Paper in structural engineering.

Khaled was highly respected by his students and colleagues and many wished to express their feelings.

"There are moments in our lives when we feel helpless in front of the scale of sadness that conquers our heart and mind. This is one of them. The only relief is to cry a brilliant student, a brilliant professor and colleague, and above all a very good friend. God help his family" (Bilal Hamad).

"It is indeed very sad news. Khaled, a CEE-AUB graduate, was a true friend and colleague for most of us. We will surely miss him. May his soul rest in peace" (Muhamad Harajli).

"I am very sorry to lose a friend and member of my family. I knew Khaled for many years; I started as his academic advisor for his Ph.D. He was a model graduate student at University of Manitoba. I have traveled several times with Khaled and we have shared a hotel room attending several conferences and we became very close as colleagues and family. I have traveled with him to Lebanon and I was the first friend to have been introduced to his lovely wife. He was a sincere husband and proud father. He devoted his life to research and teaching and became a full professor in a short time. He has achieved everything he wished for with dignity. I was very sorry to hear about his battle with cancer and shared with him many times his suffering and physical pain but he did not lose his faith with God. I know now he is in peace without suffering and I will really miss him until we meet again in the peaceful land. Mary and I send our sincere condolences and love to his wife and children and will pray to God to give them peace and understanding. I will miss him tremendously" (Sami Rizkalla).

"I will always remember Khaled Soudki as an outstanding researcher, scholar, and friend. Ever since he came to Queen's from Manitoba as a postdoctoral fellow, our careers have been intertwined. His subsequent research on the repair with FRP of corrosion-damaged reinforced concrete structures has led the field and significantly advanced applications of FRP. Khaled's work in prestressing of FRP was similarly groundbreaking including the development of an anchorage system that is now widely used for FRP prestressing. He was also a great mentor and I have also been impressed by the students he has supervised. His genial but hardworking manner has inspired many others. I will always value our friendship and his passing is a great loss to the field and to us as his friends" (Mark Green).

"Khaled was one of the finest individuals I've ever known. I thought that he was an exceptionally devoted son, husband, father, and professor, and certainly dearest of friends. I am heartbroken that he is among us no more. He leaves behind a tremendous void in our lives" (Marie-Anne Erki).

"Khaled was a wonderful man with many quality friends and colleagues. We are better for having had him in our lives" (Pat Heffernan).

"I have known Khaled since he was a Ph.D. student working with Dr. Sami Rizkalla. I got to know him on a more personal basis when we co-organized a National Science Foundation workshop in Dubai in April 2005. It was a pleasure to work with him and we became very good friends. Khaled is a true gentleman and I have the utmost respect for him. I will surely miss him" (Issam harik).

"Khaled will always be remembered with fondness and appreciation for the true scholar and wonderful person that he was. A wonderful person and academic who will be missed by all of us" (Riad El-Mahaidi).

"Khaled was truly a considerate nice person in parallel to being a superb and successful scholar. It always saddened him observing some of the unhealthy competitions and behaviors that occasionally occur in academia or scientific communities. I will never forget a statement I once heard him saying: "It is ok to grow and have big ambitions, ... but leave others grow also" (Amir Fam).

"On a personal note, I greatly miss Khal. He was hired shortly before me at UW and I took great joy in trying to convert him to becoming a pavement engineer. Obviously, bridges were his passion. His strength of character will always be admired and he will be fondly remembered by all of us here" (Susan Tighe).

"It is very difficult and hard to imagine that Prof. Soudki is no longer with us. No words can help ease the pain and loss we are feeling right now. Prof. Soudki was a great person. He will be in our every prayer and thought. May Allah shower him with mercy and provide us and his family the necessary strength and patience to withstand this great loss" (Tamer El-Maaddawy).

"Khaled will be missed at all levels. He will be missed as a professor, as a supervisor, and as a friend. He was the nicest person I have ever met. When I remember the small details that I lived with Khaled as his graduate student, I feel that I wanna cry! I cannot imagine going to Waterloo without passing by his office or his house. I will never forget that Khaled was always there whenever I asked for help or assistance. His last advice to me was that "life is short". That was when I visited him for the last time in the hospital. He will always be remembered as one of the best persons I have ever met" (Ahmed El-Refai).

"God knows how much we are sad regarding his death. Whatever we write about Khaled, we will never give him what he deserves. He was a brother and a friend, and I do not remember that he put someone down. He never made anyone upset even if he has the reason to do. We will never forget the moments when he gathered all of us in house and celebrated the holy Eid as we were his family members. At the end, everyone will die, but some of us will die with a goodwill as Khaled Soudki. May Allah bless his soul. I will never stop praying for him. We should always remember that we are as human being we have a mercy on people, never mind Allah who is greatest and the most merciful on his worshippers" (Moataz Badwi).

"I had the privilege of working with Khal over the last four years as a PhD student. No student could have asked for more from a graduate supervisor; he was a mentor, a colleague and friend in one. He was always professional, yet always personable, genuine and supportive. Khal was rare in that he had high intellect and achieved much success, yet remained humble to a fault. I hold him in the highest regard, both personally and professionally, and will miss him greatly. I am proud to have had the opportunity to learn from his example" (Martin Noël).

"I was one of the lucky people to work closely with Professor Soudki for about 7 years as his student then his postdoc. He was a very knowledgeable talented professor. More importantly, from my first day working with Dr. Soudki, I found him to be one of the finest people one could ever meet; a very decent caring loving compassionate and polite person. Even during his illness, he was still showing his care, support and love to all his students. He was always there for me, during the bad times and the good times. He was a vivid example of what a good heart means. His loss is irreplaceable. Today, I mourn the loss of a true friend and a brother I never had. Dr. Soudki, words are not enough to express my sorrow for losing you. You are leaving a very big space but you will always be remembered with all the good laughs, thoughtful actions and good memories" (Noran Wahab).

Khaled, will surely be missed greatly by all of us and you will always be remembered.

Rest in peace.

A Symposium Honoring Khaled Soudki

Towards Sustainable Infrastructure with Fiber Reinforced Polymer Composites

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Preface

The use of Fiber-reinforced polymer (FRP) composite materials in new construction and repair of concrete structures has been growing rapidly in recent years. FRP provides options and benefits not available using traditional materials. The promise of FRP materials lies in their high-strength, lightweight, noncorrosive, nonconducting, and nonmagnetic properties. ACI Committee 440 has published several guides providing recommendations for the use of FRP materials based on available test data, technical reports, and field applications. The aim of these document is to help practitioners implement FRP technology while providing testimony that design and construction with FRP materials systems is rapidly moving from emerging to mainstream technology.

In addition to this publication, ACI Committee 440 organized four special technical sessions entitled “Towards Sustainable Infrastructure with Fiber Reinforced Polymer Composites,” two sessions at the ACI Fall 2014 Convention in Washington, DC, on October 26, 2014, and two sessions at the ACI Spring 2015 Convention in Kansas City, MO, on April 13, 2015. These sessions provided a worldwide state-of-the-art forum for researchers, civil/structural engineers, contractors, consultants, practitioners, and regulatory authorities to exchange recent advances in both research and practice, and to share information, experience, and knowledge in the implementation of FRP technology. The technical papers presented at the sessions and published in this volume included the most recent analytical and experimental research work as well as selected field applications, design, and construction guidelines. The sessions were well attended and generated substantial technical discussion and exchange of new technology. Attendees were able to:

- a) Learn about the wide usage of FRP in new construction and repair of concrete structures
- b) Understand the design process available in several design guidelines that provides recommendations for the use of FRP materials based on available test data, technical reports, and field applications
- c) Demonstrate how to evaluate existing structures prior to strengthen/rehabilitation using FRP
- d) Recognize examples of the types of evaluation that can be performed on existing structures

This Special Publication consists of a total of 22 papers, of which 13 papers were presented in the four special sessions sponsored by ACI Committee 440 at the ACI Fall 2014 and the ACI Spring 2015 conventions. These papers are SP-322—3, SP-322—4, SP-322—5, SP-322—9, SP-322—10, SP-322—11, SP-322—12, SP-322—13, SP-322—14, SP-322—15, SP-322—16, SP-322—17 and SP-322—18.

In addition, the following 19 presentations were presented in the four sessions, however, and were not submitted for publication:

1. FRP for Sustainable Precast Concrete Double Tees
By: Sami Rizkalla, North Carolina State University, USA
2. Fibre-Reinforced Cementitious Matrix Systems for Shear Strengthening of RC Beams
By: Jeffrey West, University of Waterloo, Waterloo, ON, Canada
3. Effect of FRP Repair on the Bond Behavior of Corroded Reinforced Concrete Beams
By: Rania Al-Hammoud, University of Minnesota Duluth, USA
4. Corrosion in Reinforced Concrete Columns Wrapped with Carbon Fiber Reinforced Polymer Sheets
By: Mark Green, Queen’s University, Kingston, ON, Canada
5. Development Length of CFRP Rods Bonded to Concrete
By: Issam Harik, University of Kentucky, Lexington, KY, USA

6. Transfer and Development Lengths of Prestressed CFRP Bars in SCC
By: Slamah Krem, University of Waterloo, Waterloo, ON, Canada
7. Innovative FRP-Reinforced Decks for Movable Bridges
By: Amir Mirmiran, Florida International University, USA
8. Effect of Environmental Exposure on Hybrid Composite Beam Bridge Shell Elements
By: John Myers, Missouri S&T, Rolla, MO, USA
9. Strengthening Using FRP Composite: Case Studies for Design and Construction Consideration
By: Tarek Alkhrdaji, Structural Technologies, Columbia, MD, USA
10. Mechanochemistry of NSM CFRP-concrete Interface in Thermally Conductive Distress
By: Yail Jimmy Kim, University of Colorado Denver, Denver, CO, USA
11. Utilizing FRP Systems for Retrofitting Existing Reinforced Concrete Buildings to Mitigate Against Progressive Collapse
By: Khaled A. El-Domiaty, Stone Security Engineering, Arlington, VA, USA
12. Shear Behavior of Concrete Masonry Beams Reinforced with FRP Reinforcing Bars
By: Ted Sherwood, Carleton University, Ottawa, ON, Canada
13. A Study of Recovery Stresses Generated by NiTi Shape Memory Alloy Wires in CFRP/SMA Patches
By: Riadh Al-Mahaidi and Abdul Jabbar Abdy, Swinburne University, Australia
14. Life-Cycle-Based Environmental Assessment of Different Shear Strengthening Options for RC Bridge Piers
By: Andrea Prota, Costantino Menna, Loredana Napolano, and Domenico Asprone, University of Naples, Naples, Italy
15. Coupled EBNSM Strengthening for Concrete T- and Rectangular Beams with Corrosion Exposure
By: Hayder A. Rasheed, Kansas State University, Manhattan, KS, USA and Augustine F. Wuertz, White Engineering Associates, USA
16. Effects of Concrete Steam-Curing on the Mechanical Behavior of CFRP Bars—9:45 am
By: Jonathon D. Tanks and Stephen R. Sharp Virginia Center for Transportation Innovation & Research, Charlottesville, VA, USA
17. Advances in FRP Products for Infrastructure Renovation
By: Mohammad R. Ehsani, QuakeWrap Inc., Tucson, AZ, USA
18. Numerical Comparison between TRM and FRP in Strengthening Corroded Reinforced Concrete Beams
By: Ahmad A. Rteil and PKM Moniruzzaman, University of British Columbia, Kelowna, BC, Canada
19. FRP-Strengthened Reinforced Concrete Beams with Basalt Composite Sheets
By: Rami A. Hawileh, American University of Sharjah, Sharjah, United Arab Emirates

In accordance with the standard review procedures established by ACI, all papers were reviewed by at least four experts in the subject area and approved for publication. All submitted papers were given serious consideration before a decision regarding publication was made.

This volume could not have been put together without the dedication, cooperation, and assistance of many volunteers and ACI staff members. First, we would like to thank the authors for meeting our various deadlines for submission. We are also grateful to the many distinguished reviewers and members of ACI Committee 440 who have carefully evaluated and thoroughly reviewed the technical papers, and whose input and advice have been a contributing factor to the success of this volume.

Last, but not least, this symposium is dedicated to the memory of Professor Khaled A. Soudki and I am pleased to offer this symposium as a tribute to an internationally distinguished researcher in the field of using FRP in concrete structures. Khaled leaves behind a legacy of scientific research all over the world. He will always be remembered for his contribution to civil engineering, and we will all miss him dearly.

Editor

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Effect of FRP Wrapping Stiffness and Concrete Cover on Lap Splice Failure Behavior

Rayed Alyousef, Timothy Topper, and Adil Al-Mayah

Synopsis: Lap splices are an easy to implement low cost method of transferring force between reinforcing bars in concrete structures. However, the bond between lap spliced bars is usually the weakest region in a reinforced concrete structure. Fiber reinforced polymer materials (FRP) are widely used to strengthen and repair lap splices because of their high strength, durability and ease of handling. Researchers have found that increased concrete cover provides an increase in bond strength similar to that supplied by wrapping with FRP sheets. Currently the FRP industry produces a new generation of high stiffness FRP sheets that provide a high degree of confinement and large increases in bond strength to lap splices.

This paper compares the effectiveness of wrapping with very high stiffness carbon FRP sheets (CFRP 900), wrapping with low stiffness glass FRP sheets (GFRP 430) and no wrapping on the bond strength of lap splice connections for various concrete covers. The test variables were the amount of concrete cover and the wrapping condition. The results showed that the GFRP wrapped beams had an increased in bond strength of approximately 25% compared to the unwrapped beams for each of the concrete covers. However, the CFRP wrapped beams had a percentage increase in bond strength that decreased as the concrete cover increased. The CFRP wrapped beams had increases in bond strength of 71%, 60% and 44% compared to the unwrapped beams for concrete covers of 20mm, 30mm and 50 mm, respectively.

Keywords: bond behavior, thickness of concrete cover, GFRP and CFRP strengthening and lap splice.

Rayed Alyousef is a PhD candidate at the University of Waterloo, Canada, and a Lecturer (on leave), Dept. of Civil Engineering, Prince Sattam Bin Abdulaziz University, Al-Kharj, Saudi Arabia. His MASc was received from the University of Leeds in the United Kingdom in 2011 and his BEng was received from the King Saud University in Saudi Arabia in 2008. His research interests include the fatigue behavior of concrete members, and the rehabilitation of concrete structures using fiber reinforced polymer composites.

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Adil Al-Mayah is an assistant professor in the Department of Civil and Environmental Engineering/Mechanical and Mechatronics Engineering (cross appointment) at the University of Waterloo, Waterloo, Ontario Canada. His research includes fiber reinforced polymer composite mechanics and applications in the repair and strengthening of concrete structures, imaging-mechanics integration, and the numerical and mathematical modeling of contact surfaces.

INTRODUCTION

General

A lap splice is commonly used to transfer force between steel bars in a reinforced concrete member. It is a simple, effective and cost efficient connection method. Failure of the bond of steel rebars in the absence of stirrups often occurs by splitting of the concrete cover on the tension face or side face of a beam that affects the serviceability of reinforced concrete structures. Moreover, the lap splice connection is usually the weakest link in reinforced concrete structures because it is weak in bond. During monotonic loading, bond deterioration of a concrete member results in an increase in deflection, an increase in the number of cracks, an increase crack widths and a decrease load carrying capacity, which can lead to a brittle failure. The main factors that affect the bond strength of a lap splice are the concrete cover, the lap splice length, the concrete compressive strength and the confinement (stirrups or FRP) (Orangun et al. 1977; Darwin et al. 1996a; Fib 2000; ACI 2008; ACI 2012). The bond failure of the lap splice can occur by splitting or pullout (Orangun et al. 1977; Eligehausen et al. 1983; FIP 2000; ACI 2003). Pullout failure happens when the bond strength of the lap splice is high. This failure occurs by a shearing off of the steel bar ribs from the surrounding concrete (ACI 408, 2003). On the other hand, a splitting failure takes place when the confinement of the steel bar is insufficient and the concrete cover is small resulting in the development of splitting cracks at the steel bar rib (ACI 408, 2003). The splitting cracks are parallel to the steel bar at the thinner of the side or bottom concrete covers or the half of the spacing between the steel bars (ACI 408, 2012). A bond pullout failure is preferable because it is gradual in nature and provides a warning of failure in the form of an increase in deflection before full failure occurs, unlike a splitting failure that occurs suddenly.

Effect of thickness of the Concrete Cover on the Bond Strength of a Lap Splice

The concrete cover is the clear distance from a steel reinforcing bar to an external surface of a beam. The smaller of the side or bottom concrete cover thicknesses governs the splitting failure for a given lap splice length and confinement level. The mode of failure for the lap splice has been found to be strongly affected by the thickness of the concrete cover (Tepfers 1973; Orangun et al. 1977; Darwin et al. 1996a; ACI 2003 and ACI 2012). For a lap splice without stirrups or FRP confinement, the thickness of the concrete cover and the concrete tensile strength govern the splitting bond strength capacity (ACI 2003). Increasing the thickness of the concrete cover leads to an increase in the force transfer between the concrete and the steel bar resulting in a higher bond strength. The relationship between the concrete cover and the bond strength is not linear, because the distribution of the tensile strength of the concrete surrounding the lap splice is not constant (Canbay and Frosch 2005). When the concrete cover or bar spacing is small, a splitting tensile failure takes place at short anchorage lengths. However, for a large concrete cover, a pullout failure may occur. The pullout failure can happen with some splitting cracks if the bond length has higher confinement (stirrup or FRP sheet) along the steel bar. The splitting bond failure results in a lower bond strength than the pullout bond failure (ACI 408, 2003). The confinement effect produced by using FRP wrapping is expected to decrease with an increasing concrete cover because the confinement stresses in the concrete adjacent to the steel bars will be reduced as the cover thickness increases. Furthermore, Canbay and Frosch (2005) studied the relationship between the concrete

cover and the bond strength b_s and found that the bond strength varies with the square root of the product of the minimum clear concrete cover (c) and the lap splice diameter (d_c) $b_s = \sqrt{c/d_c}$.

Effect of FRP Wrapping on Bond Strength

Many of the old reinforced concrete structures need strengthening due to an increasing live load, a change of use of a structure and/or its deterioration. Among many methods of repairing or strengthening of existing structures, wrapping with fiber reinforced polymer (FRP) composites is inexpensive and easy to execute. The FRP materials have a high strength to weight ratio, a high fatigue strength, and a good durability. In addition, they are corrosion free materials, and easy to handle and apply to the external surface of a reinforced concrete member (ACI 2008). The FRP wrapping along the lap splice increases the confinement stress on the bar. The improvement of the bond strength due to the external FRP confinement is independent of the ultimate strength of the FRP sheets but proportional to their modulus of elasticity (Harajli et al. 2004). Hamad et al. (2004) studied the effect of GFRP sheets on the bond strength along a lap splice and found that the GFRP sheet increased the bond strength, the peak load and the deflection at the peak load of a beam compared to these values for an unwrapped beam. For the same area of material, the use of external FRP confinement is more efficient and effective than stirrups in controlling splitting cracks (Hamad et al. 2004). The FRP wrapping along the lap splice led to a decrease in the width of the splitting cracks and a delay in the propagation of the splitting cracks compared to an unwrapped beam (Garcia et al. 2014). Shihata (2011) studied the effect of low stiffness CFRP sheets on the bond strength of a corroded lap spliced beam and found that the CFRP sheet increased the ultimate load and the ductility compared to those of an unwrapped beam. The bond strength, the deflection and the peak load of a beam increased as the amount of the external confinement increased (Hamad and Rteil, 2006; Alyousef et al., 2015; Alyousef et al., 2016). Soudki and Sherwood (2003) studied the ability of a CFRP wrapping to increase the bond strength of a reinforcing bar as the clear concrete cover was changed. Their conclusion was that the effect of CFRP wrap for a small concrete cover was greater than that for a larger cover. The bond strength increased by 21% and 3% for concrete covers 15 mm [0.6 in.] and 30 mm [1.2 in.], respectively. In addition, the failure mode for both specimens changed from a splitting failure to a pullout failure. Bousias et al. (2007) reported that in beams reinforced with varying numbers of FRP layers the increase in load capacity was not proportional to the number of layers.

Effect of FRP Wrapping on Slip Behavior

Harajli et al. (2002, 2004) and Harajli (2009) studied the effect of FRP wrapping on the bond stress-slip behavior of a beam that experienced a splitting bond failure under monotonic load. Four stages of slip before failure were reported, as shown in Fig. 1. In the first stage, the stiffness of the bond stress versus slip relationship is similar to that for a pullout failure. The bond resistance in this first stage is attributed to the chemical adhesion and friction between the bar and the concrete and lasts until tensile cracks develop in the concrete. The second stage begins as the tensile cracks develop and is characterized by a bond stress versus slip curve that is lower than that for a pullout failure and continues until the cracks propagate to the surface. Then a third stage begins which is characterized by a sudden and rapid drop in the bond stress and is usually considered to be a bond failure. Finally, there is a fourth stage characterized by a continuing decrease in bond stress and an increase in slip.

RESEARCH SIGNIFICANCE

Many of reinforced concrete structures built before the introduction of the present bond code need to be strengthened because of a change of use or an increase in their live load. The FRP industry has increased the modulus of FRP sheet which is used for the repair and strengthening of lap splices in beams. Some researchers have studied the effect of FRP sheet on the bond strength of a lap splice. However, the studies have not used the new high stiffness CFRP sheet. Also the thickness of concrete cover, as another factor influencing the strength of lap splices, has not been studied in conjunction with high stiffness FRP sheets strengthening. To address this gap, this paper studies the effect of the lowest and highest FRP sheet available combined with various thicknesses of the concrete cover to examine the effectiveness of FRP wrapping and concrete cover on the bond strength of a lap splice.

EXPERIMENTAL PROGRAM

General

The main goal of this study is to investigate the improvement in lap splice strength provided by low and high stiffness FRP wrappings and various concrete cover thicknesses. Nine reinforced concrete beams containing lap splices in their mid-span were constructed and tested under a monotonic loading. The nine beams were divided into three groups A, B and C. The test variables are shown in Table 1.

Test Specimen

All beams were tested monotonically under four-point loading that produced a constant moment region at the mid span. All beams had the same dimensions; 2200 mm [86 in.] in length, 250 mm [10 in.] in width, and 350 mm [14 in.] in depth. The distance between the two supports was 1800 mm [72 in.] divided into three equal length regions, two shear span regions and a constant moment region containing the lap splice. The lap splice length was 300 mm [12 in.] to ensure that a bond splitting failure occurred before the steel stress reached the yield strength. Each beam was reinforced with two 20M [No. 6] steel rebars spliced in the mid span. Also, each beam was reinforced with two 10M [No. 3] steel bar at the compression zone outside the constant moment region. Moreover, for each beam, transverse reinforcement was used in the shear span at a spacing of 100 mm [4 in.] to avoid a shear failure. The beams cross sections and reinforcing details for Group B (30 mm concrete cover) and Group C (50 mm concrete cover) were the same, as shown in Fig. 2. However, Group A beams had steel bars that were de-bonded by Polyvinyl chloride (PVC) tube from the edge of the lap splice zone to the end of the beam. These bars were used to measure the bond slip, as shown in Fig. 3.

FRP strengthening

One layer of U-shaped FRP sheets 950 mm [38 in.] length were used to cover the constant moment region. The width of the FRP sheets was 900 mm [36 in.] for Group A and 600 mm [24 in.] for Groups B and C as shown in Fig. 4 and Fig. 5, respectively. Two types of FRP wrapping sheets were used in this study. Table 2 summarizes the properties of the FRP wrapping sheets. The GFRP sheet was 430 and the CFRP sheet was 900. The weights of the GFRP sheet and the CFRP sheet were 430 g/m² [1470 lb/in².] and 900 g/m² [3075 lb/in².], respectively.

Materials and Test Procedure

The nominal yield strength provided by the supplier for the 20M [No. 6] deformed steel was 450 MPa [65 ksi]. Each group of beams was cast separately. The average concrete compressive strength was measured by testing four 100 mm × 200 mm [4 in. × 8 in.] cylinders at various ages as shown in Fig. 6. The average concrete compressive strength at 28 days for groups A, B and C was 42 MPa [6000 psi], 33 MPa [4800 psi] and 35 MPa [5100 psi], respectively.

The specimens were tested under a four-point bending loading. Displacement control was used until failure with a displacement rate of 0.15 mm [0.006 in.] per minute. Linear variable differential transducers (LVDT) were used to measure the mid span deflection for all groups and the slip of the steel bars for Group A.

TEST RESULTS AND DISCUSSION

General

For all the unwrapped beams, the first cracks that appeared were flexural cracks in the tension zone at both ends of the lap splice. As the load increased, a number of the flexural cracks appeared at random locations within the constant moment region. Also, the width of the flexural cracks increased. As the load increased further, flexural and shear cracks appeared in the shear span and grew to the loading point. The flexural cracks within the constant moment region propagated upward to as far as the compression zone of the beam as the applied load increased. At a still higher load level, the tensile force in the concrete surrounding the lap splice exceeded the concrete tensile strength and a splitting crack initiated from the flexural crack at one end of the lap splice on the bottom side of the concrete cover. The splitting crack grew longitudinally parallel to the lap splice. When the bond between the lap splice and the

surrounding concrete was lost due to the splitting cracks, failure happened suddenly followed by a rapid decrease in the beam's load carrying capacity. There was no warning such as an increase in the deflection before the failure took place. A post-failure investigation found that the width of the splitting cracks was large and the flexural cracks reached about 60% of the beam depth. The failure produced larger chunks of concrete than those later observed for the wrapped beams. It is worth mentioning that, for the unwrapped beams, the chunks spalled off the beam during loading showed no signs of abrasion in the interface of the concrete with the steel bar ribs, as shown in Fig. 7.

For the wrapped beams, the presence of the FRP sheets prevented a visual monitoring of the cracks within the constant moment region during testing. The first observed cracks were flexural cracks outside the constant moment region. As the load increased, shear cracks occurred outside the constant moment region and propagated as far as the loading point. As the load increased, the FRP sheets split vertically at both ends of the lap splice to reveal flexural cracks. The flexural cracks grew upward until they reached the compression zone. The flexural cracks within the constant moment region increased in number and width as the load increased. Compared to the unwrapped beam the presence of the FRP sheet delayed the initiation and propagation of splitting cracks.

A post-failure investigation of the GFRP wrapped beams showed that the splitting cracks were finer than cracks in the unwrapped beams. However, the widths of the flexural cracks were larger than the unwrapped beams due to the increased load. The beams failures produced relatively small chunks of concrete compared to the size of the chunks for the unwrapped beams for all concrete cover thicknesses. For the CFRP wrapped beams, a post-failure investigation revealed that the splitting cracks were the thinner than those of GFRP wrapped beams or the unwrapped beams. Fig. 8 shows the FRP wrapped beam after failure and the flexural and splitting cracks after the removal of the FRP sheets. Also, the concrete ahead of the steel bars ribs was abraded and the concrete was broken into smaller chunks for the CFRP beams than for the GFRP wrapped beams, as shown in Fig. 9.

Load Deflection Behavior

Group A (20mm [3/4 in.] Concrete Cover) – The load deflection curves obtained from the tests of group A beams are shown in Fig. 10. The stiffness for the unwrapped beam and the GFRP wrapped beam was constant from the beginning of the test until the failure and was slightly lower than that of the CFRP wrapped beam. However, after failure, the load decreased gradually for the GFRP and CFRP wrapped beams, whereas it decreased abruptly for the unwrapped beam. The FRP wrapped beams experienced increases in their ultimate loads and their deflections at their ultimate loads compared to the unwrapped beams. The failures for the unwrapped and the GFRP wrapped beams were by a splitting bond failure while the CFRP wrapped beam failed by pullout followed by a splitting bond failure. The beam wrapped with CFRP sheet had a significantly higher maximum load and maximum deflection than the other beams. The GFRP wrapped beam had an ultimate load and ultimate deflection at the ultimate load 128% and 147% higher than the unwrapped beam, respectively. The CFRP wrapping increased the ultimate load and deflection by 172% and 307% respectively compared to the unwrapped beam.

Group B (30 mm [1 1/4 in.] Concrete Cover) – The load deflection curves for the beams of group B are shown in Fig. 11. The stiffness for all beams was the same from the beginning of the test until failure. The presence of the FRP wrapping within the constant moment region did not affect the cracking moment or the flexural behavior of the beams. Failure for all the wrapped and unwrapped beams was by a splitting bond failure. The FRP wrapped beams maintained their load after failure while the load dropped suddenly for the unwrapped beams. The GFRP wrapping increased the ultimate load and deflection by 124% and 143% respectively compared to the values for the unwrapped beam. The CFRP wrapping increased the ultimate load and deflection by 160% and 210% respectively compared to the values for the unwrapped beam.

Group C (50mm [2 in.] Concrete Cover) – The load deflection curves for the group C beams are shown in Fig. 12. The stiffness of the unwrapped beam was lower than that of the FRP wrapped beams. However, the stiffness for the GFRP wrapped beam and the CFRP wrapped beam was identical from the beginning of the test until the GFRP wrapped beam failed. The FRP wrapped 50 mm [2 in.], beams had much larger deflections at failure than either the unwrapped beam or the wrapped 20 mm and 30 mm cover beams. The ultimate deflection for the CFRP wrapped beam was more than twice the ultimate deflection of the GFRP wrapped beam. The GFRP wrapping increased the ultimate load and the deflection by 127% and 238% respectively compared to the unwrapped beam. The CFRP wrapping increased the ultimate load and the deflection by 143% and 481% respectively compared to the unwrapped beam.

Bond Strength

The bond stress is the shear stress between the steel bar ribs and the surrounding concrete along the lap splice. The average bond strength can be calculated based on the steel bar force divided by the surface area of the steel bar along the lap splice length. The steel stress can be obtained directly from the strain gauge reading or from the ultimate load of the beam by using a cracked section analysis. The formula below can be used to calculate the average bond strength:

$$u = \frac{A_b f_s}{\pi d_b l} = \frac{f_s d_b}{4 l}$$

where u is the average bond strength, A_b = the bar area, f_s is the steel stress at beam failure, d_b is the steel bar diameter and l is the lap splice length.

There are different models in the literature to predict the average bond strength between the steel bar and the surrounding concrete. The Orangun et al. (1977) model is used in this paper to evaluate the actual average bond strength along the lap splice. This model is given below:

$$u = \left[0.10 + \frac{c}{4d_b} + \frac{4.2d_b}{l_d} \right] \sqrt{f'_c}$$

where c is the clear concrete cover (mm), d_b is the steel bar diameter (mm), l_d is the lap splice length (mm), f'_c is the concrete compressive strength (MPa).

Hamad et al. (2004) studied the use of a FRP sheet to improve the bond strength and added a new parameter to the bond prediction equation:

$$K_{tr,f} = \frac{C_1 * A_{tr,f} * f_{fe}}{s_f * d_b * n_b} \leq 0.25$$

where C_1 is a proportionality constant = 1/16.6, $A_{tr,f}$ is the total cross sectional area of FRP, f_{fe} is the effective stress in the FRP laminate, s_f is the width of the FRP wrap, d_b is the diameter of the steel rebar and n_b is the number of spliced bars.

Table 3 shows the average bond strength of the lap splices obtained from the tests and compares these results to the predicted results for the unwrapped and FRP wrapped beams. The prediction equations used were those proposed by Orangun et al. (1977) and Hamad et al. (2004). The ratio of the actual bond strength to the predicted value varies from 96% to 134%. The ratio of the actual bond strength to the predicted bond strength for the CFRP wrapped beams was 134%, 115% and 111% for Group A (20 mm concrete cover), Group B (30 mm concrete cover) and Group C (50 mm concrete cover), respectively. The ratio of the actual bond strength to the predicted bond strength value for the GFRP wrapped beams was 113%, 99% and 106% for Group A (20 mm concrete cover), Group B (30 mm concrete cover) and Group C (50 mm concrete cover), respectively. The limitation of 0.25 for $K_{tr,f}$ in the Hamad et al. (2004) equation gives a $K_{tr,f}$ value for the GFRP sheets of 0.14. However, the $K_{tr,f}$ value for the new high strength CFRP sheets is 0.89 which is more than three times the value using this limitation. The $K_{tr,f}$ limitation which was provided by Hamad et al. (2004) preceded the introduction of new high stiffness CFRP sheets and is too conservative for the new high strength CFRP sheet and especially so for a small concrete cover. The calculation of $K_{tr,f}$ is shown in the Appendix.

Effect of the FRP Wrapping on the Bond Strength

Table 4 shows the effect of the FRP wrapping on bond strength for the three different three concrete covers. The bond strength results were normalized to the square root of the concrete compressive strength. The increase of the bond strength due to low stiffness (GFRP 430) sheet was almost constant at 126% for the three different concrete covers. The GFRP wrapping increased the normalized bond strengths for the 20 mm [3/4 in.], 30 mm [1 1/4 in.] and 50 mm [2 in.] concrete cover beams by 128%, 124% and 127% compared to those of the unwrapped beams, respectively. However, the rate of the increase of the normalized bond strength decreased as the concrete cover

increased for the CFRP wrapped beams. The CFRP wrapping increased the normalized bond strengths for the 20 mm [3/4 in.], 30 mm [1 1/4 in.] and 50 mm [2 in.] concrete cover beams compared to those of the unwrapped beams by 171%, 160% and 144%, respectively. The normalized bond strength of all concrete covers and wrapping conditions are shown in Fig. 13.

Effect of the Concrete Cover on the Bond Strength

Table 5 shows the effect of the concrete cover on the bond strength for the three different wrapping conditions. For the unwrapped beams, the normalized bond strength increased by 5% and 41% as the concrete cover increased from 20 mm [3/4 in.], to 30 mm [1 1/4 in.] and 50 mm [2 in.], respectively. For the GFRP wrapped beams, the increases in normalized bond strength with concrete cover were similar to those of the unwrapped beams. Also, the bond strength of the 50 mm [2 in.] concrete cover CFRP wrapped beam was 18% greater than that of the beam with a 20 mm [3/4 in.] concrete cover while the bond strength of the 30 mm [1 1/4 in.] cover beam was almost the same as that of the 20 mm [3/4 in.] cover beam. The normalized bond strengths of the beams for all wrapping conditions and concrete covers are shown in Fig. 14.

Bond Stress versus Slip Response for Group A (20 mm concrete cover) beams

The local bond versus slip response was monitored only for Group A for the unwrapped, GFRP wrapped and CFRP wrapped beams as shown in Fig. 15. The FRP sheet along the lap splice delayed the growth of the splitting cracks and improved the splitting bond strength compared to the unwrapped beams. The increase in bond strength is mainly due to the confinement effect of the FRP wrapping.

Harajli et al. (2004) and Harajli (2009) proposed the following equations to predict the bond stress-slip responses for the unwrapped and FRP wrapped reinforced concrete beams:

$$u_1 = 2.57 \sqrt{f'_c}$$

$$u_{max} = 0.78 \sqrt{f'_c} \left(\frac{c + 56 \frac{E_f n_f t_f}{E_s n_s}}{d_b} \right)^{2/3} \leq u_m$$

$$u_{ps} = u_{sp} \left(0.5 + 46 \frac{E_f n_f t_f}{E_s C n_s} \right) \leq u_m$$

$$s_{max} = s_1 e^{3.3 \ln \left(\frac{u_{sp}}{u_m} \right)} + s_o \ln \left(\frac{u_m}{u_{sp}} \right)$$

where s_1 is 0.15 c_o (mm), s_2 is 0.35 c_o (mm), s_3 is c_o (mm) and s_o is 0.15 (mm) for the unwrapped and 0.2 (mm) for the FRP wrapped beam. Where u_1 is the maximum bond stress for a pullout failure (MPa), f'_c is the concrete compressive strength (MPa), u_{max} is the maximum bond splitting failure stress (MPa), c is the clear concrete cover (mm), E_f is the modulus of elasticity of the FRP (MPa), n_f is the number of FRP layers, t_f is the thickness of an FRP sheet (mm), E_s is the modulus of elasticity of steel (MPa), n_s is the number of bars being spliced, d_b is the steel bar diameter (mm) and c_o is the clear distance between the ribs of the steel. If the steel bar information is not available, s_1 , s_2 and s_3 can be taken to be equal to 1.5, 3.5 and 10 mm, respectively, α is 0.7 and β is 0.65. From the above equation, the type of the bond failure can be determined by calculating the value of the bond splitting strength (u_{max} and u_{ps}) and then comparing it to the value of the maximum bond strength (u_l). A splitting bond failure is expected to occur if the value of the bond splitting strength less than the value of the maximum bond strength and vice versa.

The actual and predicted bond stress-slip curves for Group A unwrapped, GFRP wrapped and CFRP wrapped beams are shown in Fig. 16. For the unwrapped beam, the figure shows that the predicted peak bond strength is slightly higher than the actual value, and the slip at the peak bond stress is more than the predicted value. In contrast, the predicted peak bond strength for the GFRP wrapped beam is slightly lower than the test value and the slip at the peak bond stress is more than the predicted value. For the CFRP wrapped beam, the peak bond stress is almost identical to the predicted value. However, the slip at the peak bond stress is more than the predicted value. Generally, the actual

value of the peak bond stress falls close to the predicted value and the slip value at the peak stress is much higher than the predicted value.

CONCLUSIONS

The effect of two FRP wrapping lap splices combined with three different concrete cover thicknesses on failure mode, lap splice bond strength and beam load deflection behaviour were studied. Based on the results, the following conclusions can be drawn:

- The ratio of the actual bond strength to the predicted bond strength for all concrete cover and wrapping condition combinations varied from 96% to 134%.
- For the beams with a 20 mm [3/4 in.] concrete cover, the CFRP wrapped beam had a much larger deflection at failure than the GFRP wrapped and the unwrapped beam.
- For the beams with a 30 mm [1 1/4 in.] concrete cover, the stiffness of all beams was the same until failure. The bond strength and the deflection at the ultimate load increased from a minimum for the unwrapped beam to successively higher values for the GFRP and CFRP wrapped beams.
- For the beams with a 50 mm [2 in.] concrete cover, the GFRP and CFRP wrapping increased the bond strength and the deflection at the ultimate load compared to the unwrapped beam. The ultimate deflection for the CFRP wrapped beam was more than twice than the of the GFRP wrapped beam.
- The increase of the bond strength due to the low stiffness (GFRP 430) sheet was almost constant at 126% for the three different concrete covers. The CFRP wrapping increased the normalized bond strengths for the 20 mm [3/4 in.], 30 mm [1 1/4 in.] and 50 mm [2 in.] concrete cover beams compared to those of the unwrapped beams by 171%, 160% and 144%, respectively.
- The limitation of 0.25 for $K_{tr,f}$ in the Hamad et al. (2004) equation gives a $K_{tr,f}$ value for the GFRP sheets of 0.14. However, the $K_{tr,f}$ value for the new high strength CFRP sheets is 0.89 which is more than three times the value using this limitation. The $K_{tr,f}$ limitation which was provided by Hamad et al. (2004) preceded the introduction of new high stiffness CFRP sheets and is too conservative for the new high strength CFRP sheet and especially so for a small concrete cover.
- The actual value of the peak bond stress falls close to the predicted value and the slip value at the peak stress is much higher than the predicted value.

Appendix. Calculation example of $K_{tr,f}$

Table 6 shows the steps in calculation of the $K_{tr,f}$ parameter of Hamad et al. (2004) for GFRP and CFRP wrapped beams under monotonic load. The calculation procedure uses the ACI 440 (2008) formula $f_{fe}=R*f_{fu}$ to calculate the effective stress of the FRP sheet. The GFRP and CFRP wrapping sheets had the following properties: n =number of FRP sheet layers =1; w_f = width of FRP sheets = 600 mm [24 in.]; s_f = width of FRP = 600 mm [24 in.]; d_f = depth of FRP sheets = 350 mm [14 in.]; f'_c = concrete compressive strength= 33 MPa [4800 psi]; n_b = number of bars being spliced = 2 and d_b = bar diameter = 20 mm [0.8 in.]. However; they differed in the moduli of elasticity, elongation, thickness and tensile strength for the GFRP sheets they were 26.5 GPa [3.84×10^6 psi], 0.022, 0.5 mm and 537 MPa [78×10^3 psi] while for the CFRP sheets they were 100 GPa [14.5×10^6 psi], 0.011, 1.3 mm [0.052 in.] and 1120 MPa [1.62×10^5 psi], respectively.

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Table 1 — Characteristics of tested beams

Groups	Beam notation	Clear concrete cover c		Bar diameter d		c/d ratio	Concrete compressive strength f'_c		Confinement at mid span
		mm	inch	mm	inch		MPa	psi	
A	A1								Non
	A2	20	3/4	20	3/4	1	42	6000	GFRP sheet
	A3								CFRP sheet
B	B1								Non
	B2	30	1 ¼	20	3/4	1.5	33	4800	GFRP sheet
	B3								CFRP sheet
C	C1								Non
	C2	50	2	20	3/4	2.5	35	5100	GFRP sheet
	C3								CFRP sheet

Table 2 — Properties of FRP wrapped sheet

Property	Carbon fiber wrapped sheet				Glass fiber wrapped sheet			
	CFRP wrap 900C fiber properties		CFRP wrap 900C cured laminated		GFRP wrap 430G fiber properties		GFRP wrap 430G cured laminated	
	MPa / mm	ksi / in.	MPa / mm	ksi / in.	MPa / mm	ksi / in.	MPa / mm	ksi / in.
Tensile strength	3,800	550	1120	162	2,250	325	537	78
Tensile modulus	242,000	35000	100,000	14500	70,000	10000	26,493	3842
Elongation (%)	1.55	--	1.1	--	2.8	--	2.21	--
Thickness (mm)	--	--	1.3	0.051181	--	--	0.508	0.02

Table 3 — Test results and analytical predictions of bond strength

Groups	Beam notation	Actual of the bond strength		Prediction bond strength		Ratio of the Actual bond to the predicted bond %
		MPa	psi	MPa	psi	
A (20 mm [3/4 in.] concrete cover)	A1	4.46	647	4.08	592	109%
	A2	5.73	831	5.05	732	113%
	A3	7.64	1108	5.7	827	134%
B (30 mm [1 ¼ in.] concrete cover)	B1	4.14	600	4.34	629	96%
	B2	5.15	747	5.19	753	99%
	B3	6.63	962	5.77	837	115%
C (50 mm)	C1	5.73	831	5.95	863	96%
	C2	7.26	1053	6.83	991	106%

Effect of FRP Wrapping Stiffness and Concrete Cover on Lap Splice Failure Behavior

[2 in. concrete cover)	C3	8.23	1194	7.42	1076	111%
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Table 4 — Effect of FRP wrapping to the bond strength

Group	Beam notation	Bond strength		Bond/ $\sqrt{f'_c}$	Effect of wrapping %
		MPa	psi		
A	A1	4.5	653	0.69	100
	A2	5.7	827	0.88	128
	A3	7.6	1102	1.18	171
B	B1	4.1	595	0.72	100
	B2	5.2	754	0.90	124
	B3	6.6	957	1.15	160
C	C1	5.7	827	0.97	100
	C2	7.3	1059	1.23	127
	C3	8.2	1189	1.39	144

Table 5 — Effect of the concrete cover on the bond strength

Type of wrapping	Beam notation	c/d ratio	Bond strength		Bond/ $\sqrt{f'_c}$	Effect of the concrete cover %
			MPa	psi		
Unwrapped	A1	1	4.5	653	0.69	100
	B1	1.5	4.1	595	0.72	104
	C1	2.5	5.7	827	0.97	141
GFRP	A2	1	5.7	827	0.88	100
	B2	1.5	5.2	754	0.9	102
	C2	2.5	7.3	1059	1.23	140
CFRP	A3	1	7.6	1102	1.18	100
	B3	1.5	6.6	957	1.15	97
	C3	2.5	8.2	1189	1.39	118

Table 6 — The steps in calculation of the $K_{tr,f}$ parameter:

Steps:	GFRP	CFRP
Nominal strength and strain of the FRP sheet		
$f_{fu} = C_E f_{fu}$ (C_E is environmental reduction factor = 1.0 to get nominal tensile strength)	$f_{fu} = 1 * 537 = 537 \text{ MPa}$	$f_{fu} = 1 * 1120 = 1120 \text{ MPa}$
$\varepsilon_{fu} = C_E \varepsilon_{fu}$	$\varepsilon_{fu} = 1 * 0.022 = 0.022$	$\varepsilon_{fu} = 1 * 0.011 = 0.011$
I_e is active bond strength length.		
For U wraps:		
$I_e = \frac{23,300}{(n t_f E_f)^{0.58}}$	$I_e = \frac{23,300}{(1 * 0.51 * 26,500)^{0.58}} = 94 \text{ mm}$	$I_e = \frac{23,300}{(1 * 1.3 * 100,000)^{0.58}} = 25 \text{ mm}$
k_1 and k_2 are coefficients to account for concrete strength and sheet layout respectively:		
$k_1 = \left(\frac{f'_c}{27}\right)^{2/3}$	$k_1 = \left(\frac{33}{27}\right)^{2/3} = 1.14$	$k_1 = \left(\frac{33}{27}\right)^{2/3} = 1.14$
$k_2 = \left(\frac{d_f - I_e}{d_f}\right)$	$k_2 = \left(\frac{350 - 94}{350}\right) = 0.73$	$k_2 = \left(\frac{350 - 25}{350}\right) = 0.93$
Calculate stress reduction factor:		
$R = \frac{k_1 k_2 I_e}{11,900 \varepsilon_{fu}} \leq \frac{0.004}{\varepsilon_{fu}}$	$R = \frac{1.14 * 0.73 * 94}{11,900 * 0.022} = 0.29$ $\frac{0.004}{\varepsilon_{fu}} = 0.18$ $R = 0.29 \leq 0.18$ Use $R = 0.18$	$R = \frac{1.14 * 0.93 * 25}{11,900 * 0.011} = 0.20$ $\frac{0.004}{\varepsilon_{fu}} = 0.36$ $R = 0.20 \leq 0.36$ Use $R = 0.20$
Calculate effective stress of fiber reinforced polymer sheet:		
$f_{fe} = R * f_{fu}$	$f_{fe} = 0.18 * 537 = 97 \text{ MPa}$	$f_{fe} = 0.2 * 1120 = 224 \text{ MPa}$
Calculate area of fiber reinforced polymer reinforcement as follows:		
$A_{tr,f} = 2 n t_f w_f$	$A_{tr,f} = 2 * 1 * 0.51 * 600 = 612 \text{ mm}^2$	$A_{tr,f} = 2 * 1 * 1.3 * 600 = 1560 \text{ mm}^2$
Calculate $K_{tr,f}$		
$K_{tr,f} = \frac{A_{tr,f} f_{fe}}{C d_b n_b s_f} \leq 0.25$	$K_{tr,f} = \frac{612 * 97}{16.6 * 20 * 2 * 600} = 0.148$ $K_{tr,f} = 0.148$	$K_{tr,f} = \frac{1560 * 224}{16.6 * 20 * 2 * 600} = 0.89$ According to Hamad et al. (2004) $K_{tr,f}$ is limited to 0.25

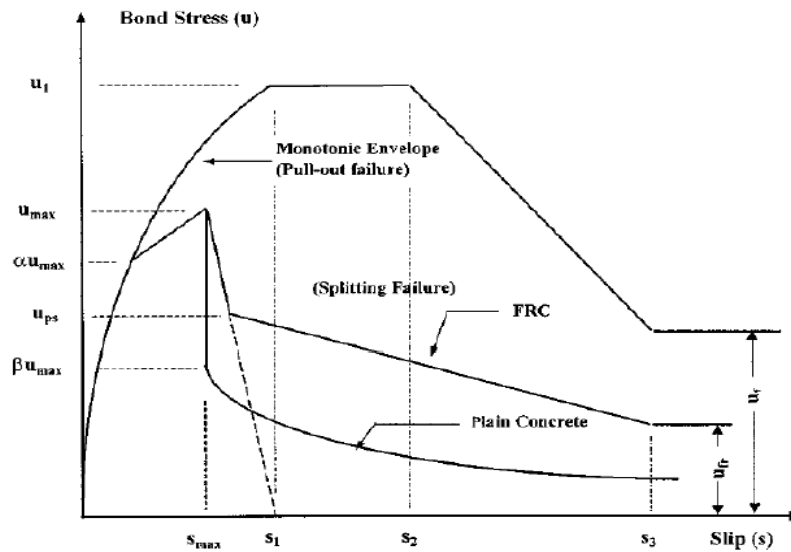


Figure 1 — General model of local bond stress-slip behavior of

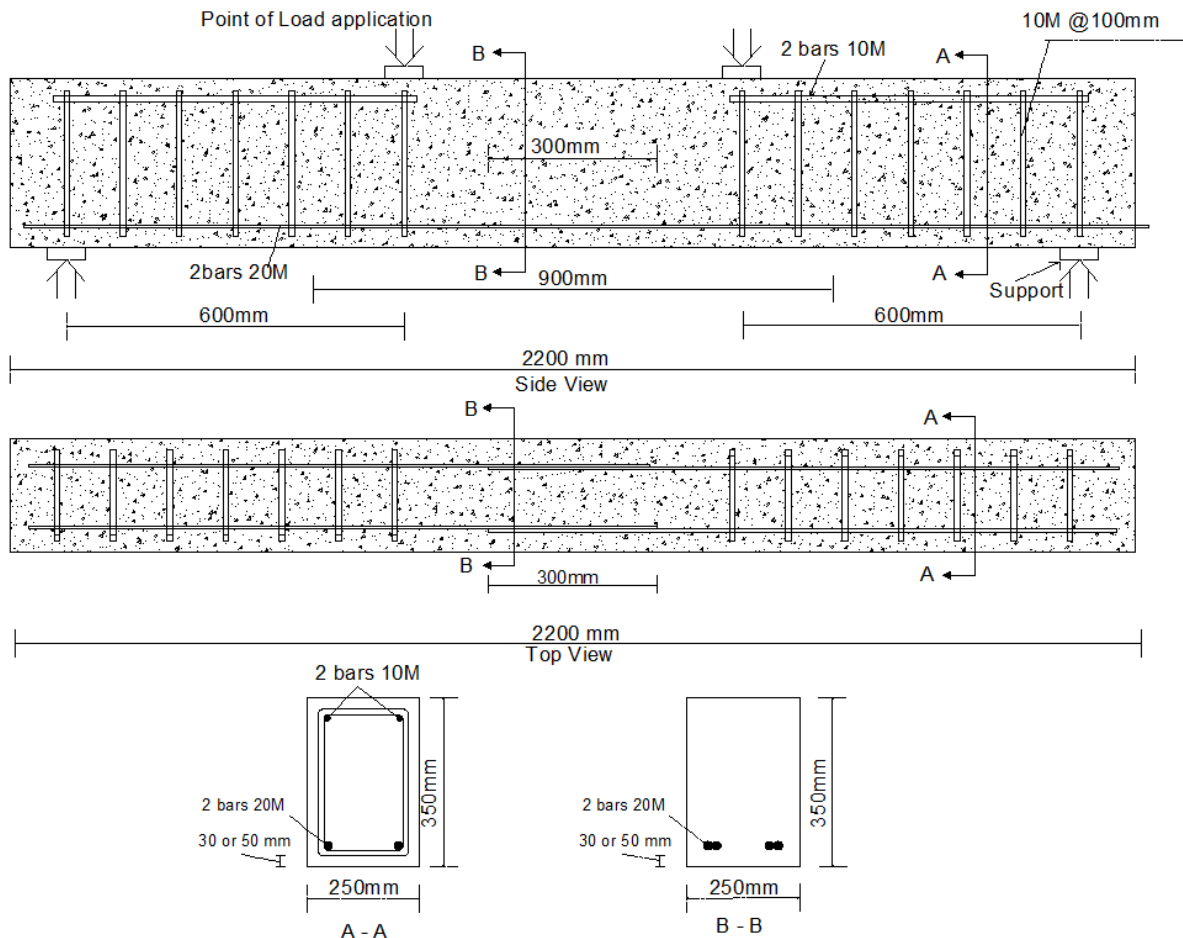


Figure 2 — Beam cross-section and reinforcement details for Group B and Group C

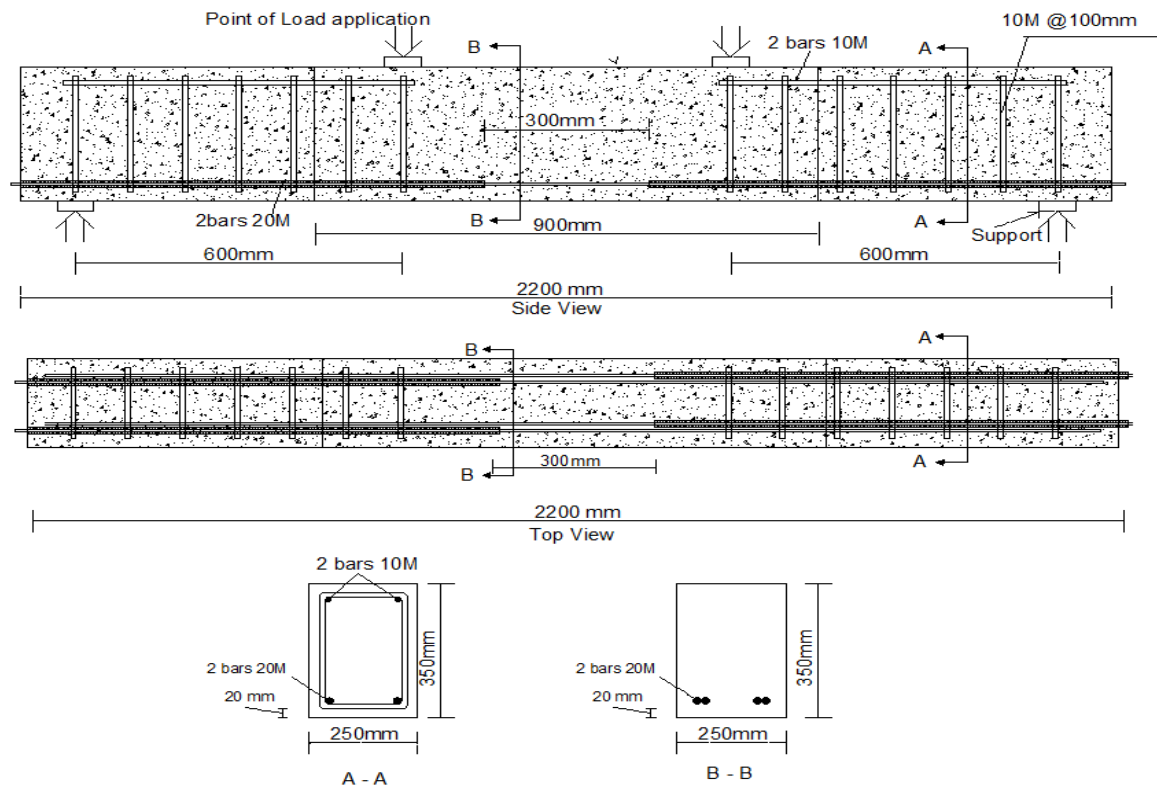


Figure 3 — Beam cross-section and reinforcement details for Group A

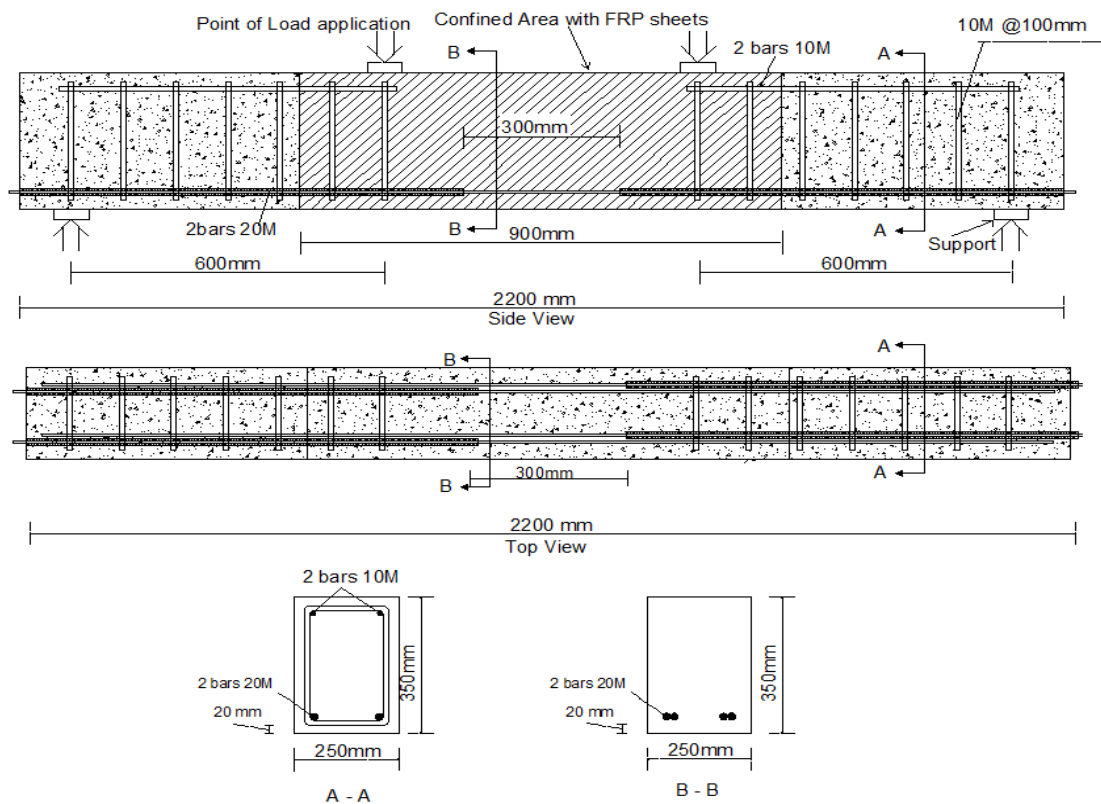


Figure 4 — FRP wrapping scheme for Group A

Effect of FRP Wrapping Stiffness and Concrete Cover on Lap Splice Failure Behavior

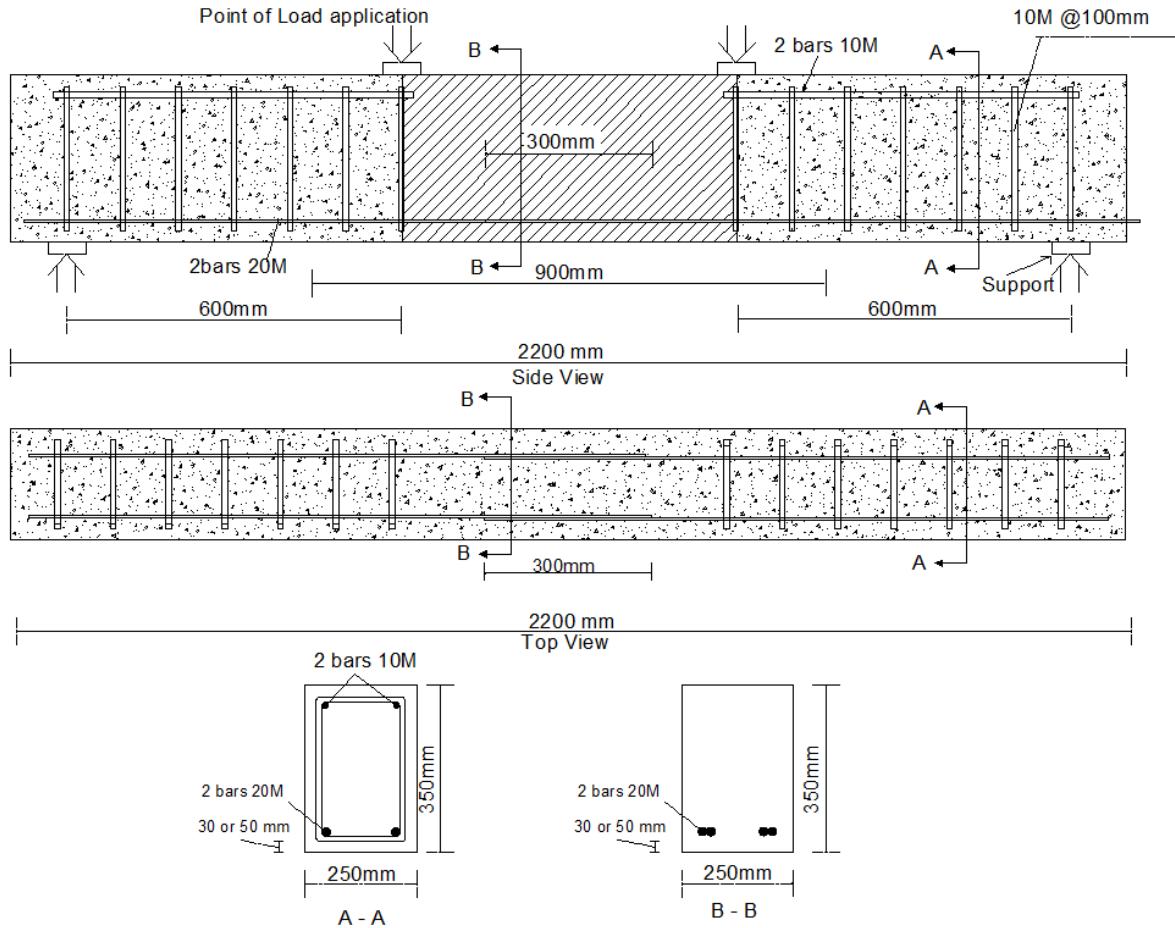


Figure 5 — FRP wrapping scheme for Group B and Group C

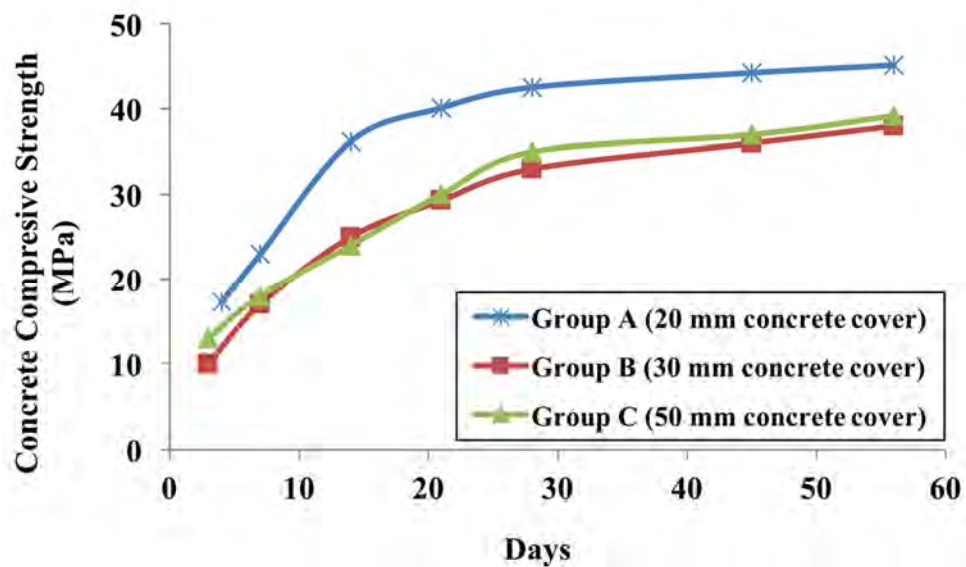


Figure 6 — Average concrete strength at different ages for all groups



Figure 7 — The interface of the concrete ahead of the steel bar ribs for the unwrapped beam



The GFRP wrapped beam after failure



The upside down beam after removing the FRP sheet

Figure 8 — The behaviour of the FRP wrapped beam after failure



Figure 9 — The interface of the concrete ahead of the steel bar ribs for the CFRP wrapped beam

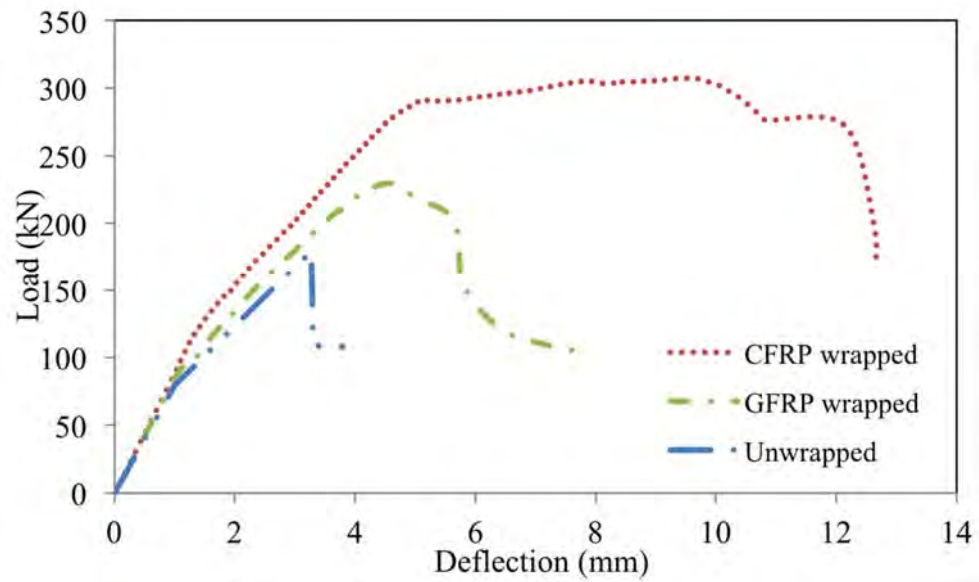


Figure 6 — Load deflection curves for group A (20 mm concrete cover)

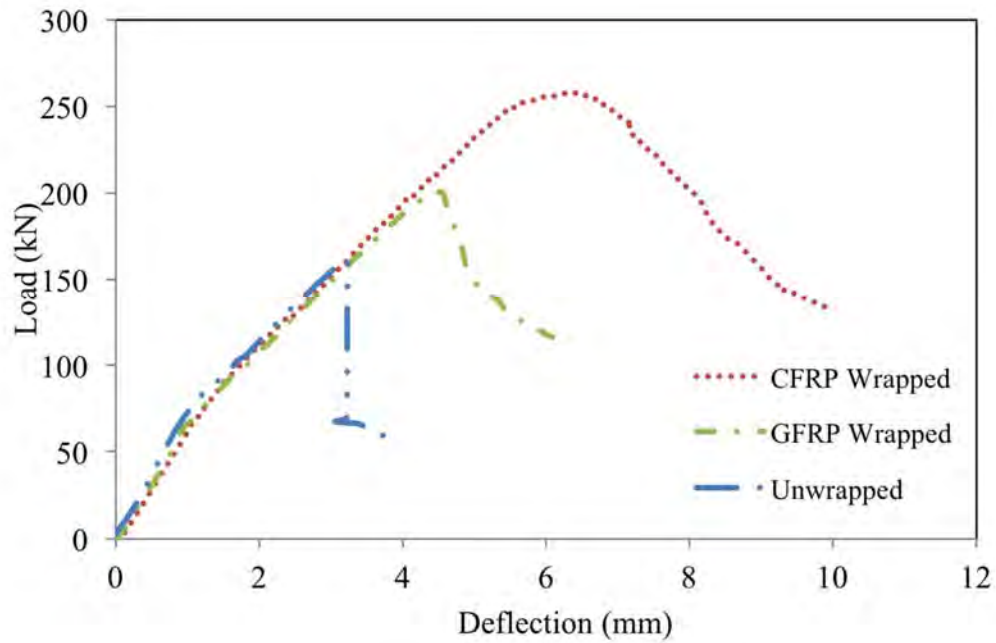


Figure 7 — Load deflection curves for group B (30 mm concrete cover)

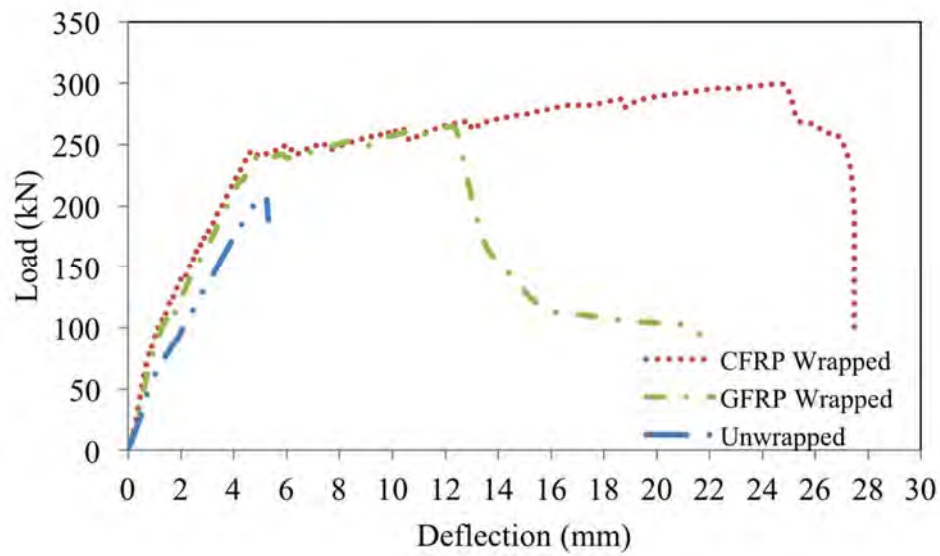


Figure 8 — Load deflection curves for group C (50 mm concrete cover)

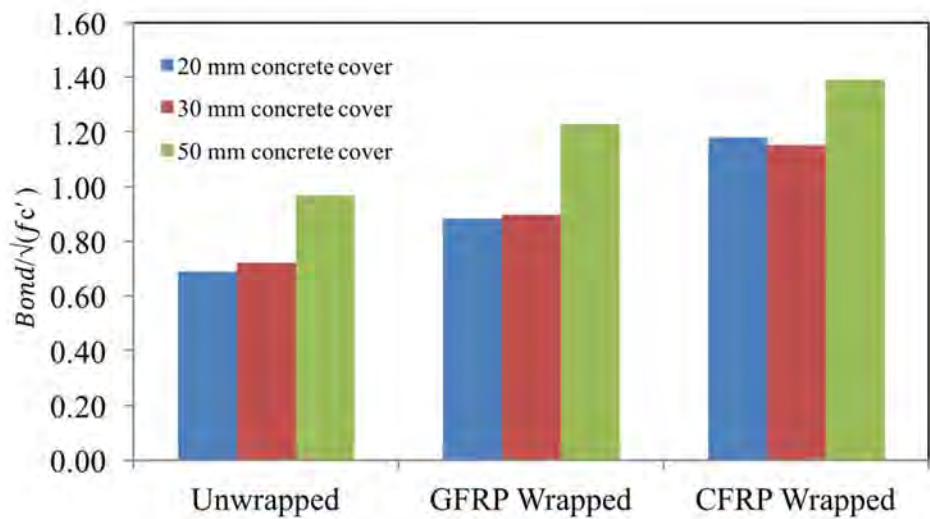


Figure 9 — Effect of the different concrete cover on the normalized bond strength for all wrapping condition

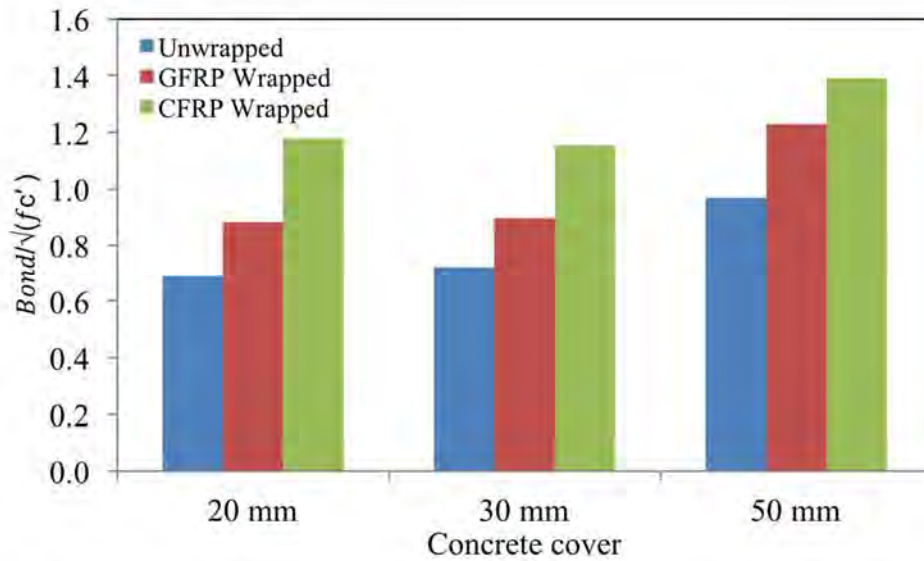


Figure 10 — Effect of the FRP wrapped on normalized bond strength at different concrete cover

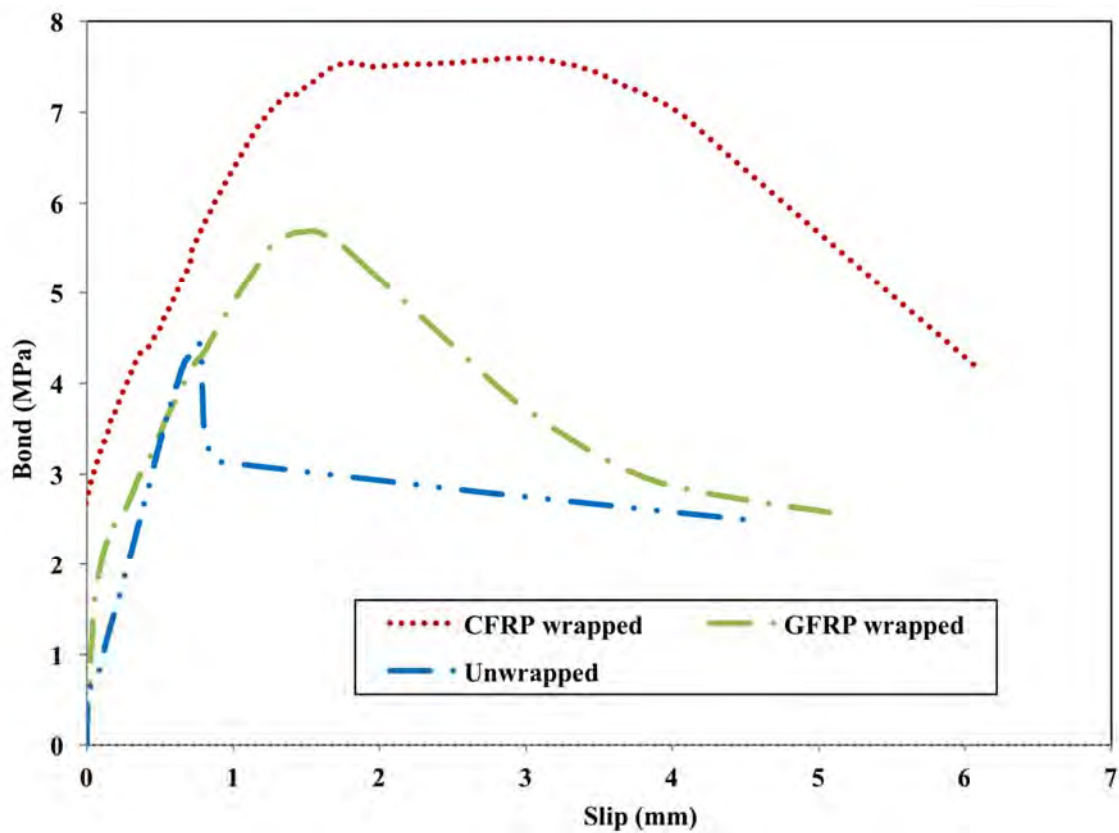


Figure 11 — The local bond stress versus slip response for Group A (20 mm concrete cover)

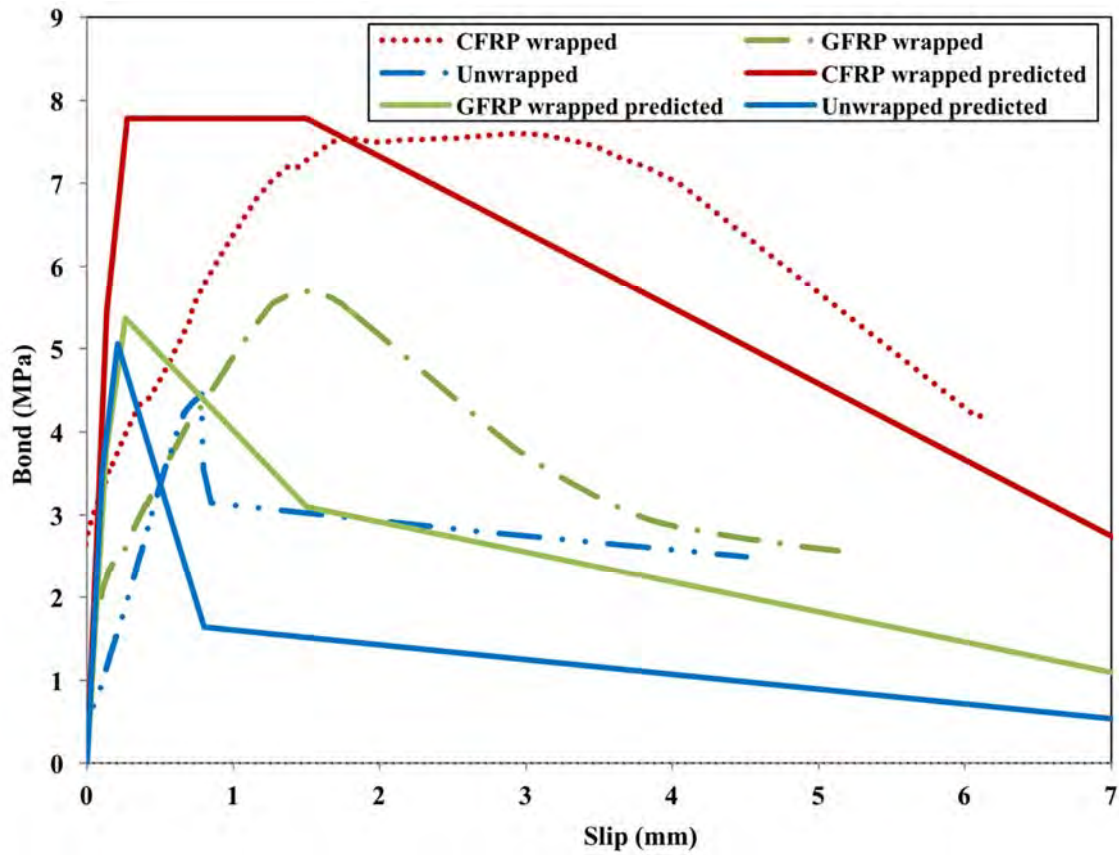


Figure 12 — The actual and predicted bond stress versus slip for Group A (20 mm concrete cover)

Shape Effect of FRP Stay-in-Place Structural Forms on Durability under Freeze-Thaw Cycles

Raouf Boles and Amir Fam

Synopsis: This study investigates the effect of aggressive regime of 300 freeze-thaw (FT) cycles, at a core temperature range of +5 °C (+41 °F) to -18 °C (-0.4 °F) on the structural behaviour and bond integrity of concrete beams cast onto glass fiber reinforced polymer (GFRP) stay-in-place (SIP) structural forms. The study aims at comparing two configurations of the SIP forms, namely a flat plate with T-shape ribs and a corrugated plate, under the potential 'frost-jacking' effect arising from FT cycles. The study explored specimens with no surface treatment, wet adhesive bonding to freshly cast concrete, and bonded coarse aggregates to enhance roughness of SIP form. It was clearly shown that flat-ribbed form specimens are superior to the corrugated form ones, as no loss in strength occurred after the FT exposure, whereas the corrugated form specimens lost 18-21%. This is attributed to the anchorage advantage provided by the T-shape rib embedment into concrete. Specimens with untreated corrugated forms showed strengths that are only 21-26% of treated ones. For flat-ribbed form specimens, the one with untreated form had 44% the strength of that with bonded aggregates.

Keywords: durability, freeze-thaw, stay-in-place forms, FRP, concrete, bond

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INTRODUCTION

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

The concept of FRP SIP forms has been studied for beam applications. A study by Honickman et al (2009) explored various bond mechanisms at the interface between glass-FRP (GFRP) flat plates and concrete slabs, namely wet adhesive bond to freshly cast concrete, a layer of coarse aggregates bonded to the plate prior to casting, and mechanical shear connectors using threaded GFRP or steel rods. The study found that adhesive bond and bonded aggregates showed superior strength and stiffness and better distribution of fine cracks. Although no slip occurred in this type of bond, sudden shear-bond failure governed at ultimate. Honickman and Fam (2009) extended this concept to a hat-shape GFRP form for girder application, where voided concrete cores were explored for box sections. Also, the behavior of the system in the negative moment region of continuous beams was investigated.

One of the most promising applications of the concept of SIP structural forms is in bridge decks due to the rapid construction feature of the system, which minimizes road closure and traffic delays. Several studies and field applications have been accomplished in this area, including static and fatigue testing (e.g. Berg et al, 2005, Nelson and Fam, 2013, Richardson et al, 2013). While durability of externally bonded FRP laminates for concrete retrofitting applications has been studied sufficiently, including exposure to FT cycles (e.g. Karbhari and Engineer, 1996, and Green et al, 2000), very limited work has been done on durability of SIP form systems. A recent study on bridge decks with GFRP SIP forms exposed to aggressive FT cycles showed insignificant reduction in ultimate strength (Boles et al, 2015). It is possible that this was attributed to the end constraints of the deck being cast on precast concrete girders with protruding stirrups embedded into the ends of the deck as in typical construction. But it is also possible that the specific SIP form geometry has a significant effect on the response of the deck to FT exposure.

RESEARCH SIGNIFICANCE

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

EXPERIMENTAL PROGRAM

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

Test Specimens and Parameters

The experimental program included nine beam specimens, 2 m (78.7 in) long each, referred to as S1 to S9 (Table 1). The beams were built using two different types of GFRP stay-in-place structural forms, namely; a flat plate with T-shape ribs, and a corrugated plate. Figure 1(a) shows the two cross-section configurations. These two types of forms were used in bridge deck applications and tested under static and fatigue loading (Nelson and Fam, 2013, Fam and Nelson, 2012, and Richardson et al, 2014). In this study, the durability of the two systems is compared using the simply supported beam specimens under four-point bending (Fig. 1(b)). This setup provides critical conditions that promotes bond and/or shear failure, as the shear span-to-effective depth ratios are 3.3-3.8 (Table 1). The objective is to examine the effects of form shape and geometry along with surface preparation technique on the strength of bond-critical specimens with and without exposure to FT cycles.

Table 1 provides a summary of test matrix. Specimens S1-S5 were control specimens tested to failure at room temperature, including the two different form types, namely corrugated plate (specimens S1-S3) and ribbed plate (S4-S5). Specimens S1-S3 had a 270x225 mm (10.6x8.8 in) cross-section, with GFRP reinforcement ratio of 3.5%, while S4-S5 had a cross-section of 290x237 mm (11.4x9.3 in), with GFRP reinforcement ratio of 2.34%, where the reinforcement ratio ρ_f is defined as the ratio of GFRP-to-concrete cross-sectional areas (A_f/bd). The two beam configurations were designed to have a similar stiffness of their reinforcement ratio ($E_f\rho_f$) of 0.67 GPa (97.2 ksi) and 0.69 GPa (100.1 ksi), for the corrugated and ribbed panels, respectively, where E_f is the GFRP longitudinal modulus. It is well established that the quantity ($E_f\rho_f$) of longitudinal reinforcement affects the shear strength of concrete beams reinforced by FRP as it directly influence the diagonal crack width, which in turn affects aggregate interlock along the inclined plane of the crack.

Different surface preparation methods were examined in both configurations, namely, ‘nothing’ other than surface cleaning of the GFRP form prior to concrete casting (specimens S1 and S4), applying an adhesive layer of epoxy shortly before casting, which bonds to concrete (S2) and applying a layer of adhesively bonded coarse aggregates to the GFRP form (S3 and S5). Specimens S6-S9 were exposed to 300 FT cycles where the core temperature range during FT cycles was +5 °C (+41 °F) to -18 °C (-0.4 °F). Specimens S6 and S7 were identical to S2 and S3, respectively, while S8 and S9 were identical to S4 and S5, respectively.

Material Properties

GFRP ribbed flat panels and corrugated panels – Commercially available pultruded GFRP panels made from E-glass fibers and polyester resin were used. The first type was 4.2 mm (0.165 in) thick flat plate with T-shaped ribs of the same thickness, spaced at 100 mm (3.94 in). The second type was 4.2 mm (0.165 in) thick corrugated panels with pin and eye end connections for splicing purposes. Five standard coupons (250x25 mm, 9.84x1.0 in) were cut from each panel, and tested in tension according to ASTM D3039/D3039M-08. Figure 2 shows the stress-strain curves of the two panels. The average tensile strength and elastic modulus of the ribbed panels were 352 MPa (51 ksi) (with a standard deviation of 16.5 MPa (2.39 ksi)) and 29.5 GPa (4278 ksi), respectively, while the tensile strength and modulus of the corrugated panels were 327 MPa (47.4 ksi) (with 15 MPa (2.18 ksi) standard deviation) and 18.9 GPa (2741 ksi), respectively.

GFRP rebar – V-Rod #2 GFRP rebar mats of 6.35 mm (0.25 in) diameter were installed in all specimens as top reinforcement for shrinkage and thermal crack control. The tensile strength and elastic modulus provided by the manufacturer were 784 MPa (114 ksi) and 46.1 GPa (6686 ksi), respectively.

Concrete – The concrete mix used in all beams had 10 mm (0.39 in) coarse aggregates and was designed to have a 7% air entrainment, a 200 mm (7.87 in) slump and compressive strength of 35 MPa (5.1 ksi). At the time of testing the control specimens, 15 concrete cylinders were tested in compression according to ASTM C39 (2010) and showed a strength range of 36.4-46.6 MPa (5.28-6.76 ksi) with an average of 41.8 MPa (6.1 ksi) and a standard deviation of

3.4 MPa (0.49 ksi). Additional 15 cylinders were exposed to the 300 FT cycles and were tested after. They showed a similar strength range of 38.2-45.9 MPa (5.54-6.66 ksi) with an average of 41.4 MPa (6 ksi) and a standard deviation of 2.05 MPa (0.297 ksi).

Coarse aggregates – 4-9 mm (0.16-0.35 in) pure silica stones were used in the ‘bonded aggregate’ system and were bonded to the GFRP panel using epoxy adhesive.

Epoxy adhesives – Two types of adhesives were used. When bonding the silica stones to the surface of the FRP panel, Sikadur 30 was used. It is a high modulus, high strength, and high viscosity epoxy paste adhesive with a 20.6 MPa (2.99 ksi) bond strength as reported by the manufacturer. For bonding freshly cast concrete to the FRP panels, Sikadur 32 Hi-Mod, a high modulus, and low viscosity epoxy adhesive with manufacturer reported bond strength of 13.1 MPa (1.9 ksi) was used to coat the surface with a thin layer, shortly before concrete casting.

Fabrication of Test Specimens

The GFRP panels were cut to the design length of specimens, 2 m (78.7 in) each. The panel surface was cleaned with acetone prior to applying the specific surface preparation and bond technique. For the specimens without surface preparation (Fig. 3(a)), concrete was cast directly onto the forms. For the bonded aggregates surface treatment, a layer of the viscous epoxy resin was applied to the horizontal surface only (i.e. the bottom flange) of the GFRP panels, followed by a layer of the coarse aggregates pressed into the adhesive layer (Fig. 3(b)). From a practical standpoint, it would be difficult to bond the aggregates to vertical or inclined surfaces. Once the epoxy has cured, concrete was cast onto the prepared SIP forms. For the wet adhesive bond system, the thin epoxy coating was applied to the entire surface of the GFRP form using a conventional brush, like paint (Fig. 3(c)). The concrete was then cast onto the forms within 15 minutes to bond to the wet adhesive. In all specimens, prior to concrete casting, additional formwork was necessary to build up the cross-section shape of the specimens. Thermocouple gauges were also secured in place at the center of the core of the specimens before casting and the top reinforcement mesh was also secured in place. After concrete hardening, the specimens were moist-cured for seven days.

Environmental Chamber Setup and Instrumentation

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

The freeze-thaw procedure used was guided by ASTM C666-97 standard. All specimens were saturated by submersion in water for at least 24 hours first, before applying the 300 FT cycles. The target range of temperatures of the concrete core during FT cycles was +5 °C (+41 °F) to -18 °C (-0.4 °F). Thermocouples were installed in the core of the specimens to monitor their temperature continuously. A typical cycle took 9 hours and 20 minutes, including about 7 hours freezing in air and about 2 hours thawing in water (Fig. 4(b)). In order to achieve the desired core temperatures, the air temperature range which is monitored through the environmental chamber controls had to be larger, +13 °C (+55.4 °F) to -25 °C (-13 °F) (Fig. 4(b)). After the specimens completed the FT cycles, they were removed from the tank, left to dry, and then tested to failure in a special setup, discussed next.

Structural Testing Setup and Instrumentation

Figure 1(b) shows a schematic of the four-point bending test setup. Pictures of the setup are also shown in Fig. 5(a) for the specimens with ribbed flat plate and in Fig. 5(b) for the specimens with corrugated plate. The simply supported span and constant moment zones were 1800 mm (70.87 in) and 300 mm (11.81 in), respectively. The specimens were tested to failure under stroke control using a 900 kN (202 kips) Reihle testing machine at a rate of 1 mm/min (0.039 in/min). One Linear Potentiometer (LP) was used at mid-span to measure the deflection. Also, two LPs were installed horizontally at each end of the specimen to measure any relative slip between the GFRP plate and concrete. Longitudinal strains were measured at mid-span, on both the bottom surface of the GFRP form, and the top surface of concrete, using 5 mm (0.197 in) and 65 mm (2.56 in) electric resistance strain gauges, respectively. Additional 100 mm (3.94 in) displacement transducers (PI gauges) were used to verify concrete strain at top surface.

EXPERIMENTAL RESULTS

Table 2 provides a summary of test results of all specimens, in terms of peak loads, maximum strains in GFRP and concrete, mid-span deflections at peak loads, load at 0.1 mm (0.0039 in) slip, and slip values at peak loads. Figure 6 shows the load-mid-span deflection responses. Figures 7 and 8 show the load-strain responses of GFRP form in tension and concrete in compression, respectively. Figure 9 shows the load-slip responses between the GFRP form and concrete. Fig. 10 shows the failure modes. The following sections discuss in detail the test results.

Effect of Surface Treatment of GFRP Forms in Control Specimens

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

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

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

Specimen S5, on the other hand, showed a considerably higher, 2.3 times, ultimate load than S4 but did not show the apparent pseudo-ductility that S4 showed, due to a brittle concrete shear failure. Figure 7(b) shows that the maximum GFRP tensile strain reached was about 3500 micro-strain only, significantly lower than the ultimate value (Fig. 2). Figure 8(b) also shows that the maximum concrete compressive strain was about -1000 micro-strain only.

Failure modes – Specimen S1 with no bond failed immediately upon first cracking as debonding occurred (Fig. 10(a)). The specimen basically broke completely into two pieces. Specimens S2 and S3 with wet adhesive and bonded aggregates exhibited flexural and flexural-shear cracks. Eventually they both failed in concrete shear through diagonal tension cracking (Fig. 10(b)). A secondary debonding failure of the sides of the GFRP form also occurred. Specimen S4 with no surface treatment exhibited one flexural crack and eventually failed by horizontal shearing of one of the

GFRP T-shape ribs from the flat GFRP base (Fig. 10(c)). Specimen 5 with bonded aggregates exhibited several flexural and flexural-shear cracks. Eventually it failed in shear by diagonal tension cracking of concrete (Fig. 10(d)). No bond failure was observed in this specimen.

Effect of Freeze-Thaw (FT) Cycles

Corrugated forms – Specimens S6 and S7 with wet adhesive and bonded aggregates interfaces, respectively, were exposed to 300 FT cycles then tested to failure after. Figure 6(a) shows that FT exposure has resulted in 21% and 18.4% reduction in the strength of specimens S6 and S7, respectively, relative to their control counterparts S2 and S3. The reductions appear to be similar despite the different surface treatment. On the other hand, the stiffness as represented by the slope of load-deflection curves appear unchanged after FT exposure. Figure 7(a) clearly shows that the GFRP strains of S6 are larger than S2 at any given load. In fact, the cracking load of S6 is smaller than S2. The figure also shows that S7 appears to have been already cracked before the test. It is possible that this might have occurred during lifting and handling of the beam into and out of the aging tank in the environmental chamber. Figure 9(a) shows that the relative slip at peak load was about 0.4 mm (0.016 in) for both S6 and S7, larger than their control counterparts. It is also noted that the loads corresponding to 0.1 mm (0.0039 in) slip are lower for S6 and S7 specimens, than their control counterparts S2 and S3, respectively (Table 2). This is attributed to the freeze-thaw cycles, which is believed to have weakened the interface bond between concrete and GFRP forms.

Ribbed forms – Specimens S8 and S9 with no surface treatment and bonded aggregates, respectively, were exposed to 300 FT cycles then tested to failure after. Figure 6(b) shows that the FT exposure had absolutely no effect on their ultimate strengths and stiffness. Clearly, their ultimate loads and the entire load-deflection curves match almost exactly those of their control counterparts S4 and S5. Similarly, Figs. 7(b), 8(b) and 9(b) confirm this observation of almost identical behavior of both exposed and control specimens. In fact, the loads corresponding to 0.1 mm (0.0039 in) slip, and indeed the slip at ultimate, are slightly higher for S8 and S9 specimens than their control counterparts S4 and S5, respectively (Table 2). This is likely attributed to the fact that the embedded T-shape ribs mechanically anchor the bottom plate to the concrete. As the concrete undergoes volumetric change due to long exposure to moisture and is not quite free to expand given the restraining effect of the anchored GFRP plate, it appears that an interface pressure may have been generated leading to a slightly enhanced bond. This effect never occurred in corrugated specimens S6 and S7, but to the contrary, the loads at 0.1 mm (0.0039 in) slip reduced after conditioning as concrete was not restrained by the GFRP plate.

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

SUMMARY AND CONCLUSIONS

This study examined the effect of 300 freeze-thaw cycles at a temperature range of +5 °C (+41 °F) to -18 °C (-0.4 °F) on two types of GFRP stay-in-place structural forms for concrete structures, namely a flat plate with T-shape ribs and a corrugated plate. Different surface preparation techniques were used for the forms. The following conclusions are drawn:

1. In specimens with flat forms having T-shape ribs, freeze-thaw exposure did not cause any strength reduction or change in failure modes, neither in the one without surface treatment, nor in the one with bonded aggregates. This suggests that the embedment of the T-shape ribs in concrete counteracts any ‘frost-jacking’ effect between the flat plate and concrete.
2. In specimens with corrugated forms, freeze-thaw exposure resulted in 21% and 18% reductions in strength of specimens with wet adhesive and bonded aggregates, respectively. Also, failure mode changed from concrete shear

to bond failure. Unlike ribbed panels, this system has no mechanical interlocking between the form and concrete.

3. Specimen with untreated corrugated form showed a strength that is only 21% and 26% of those treated with wet adhesive and bonded aggregates, respectively. For flat plate forms with T-shape ribs, the specimen with untreated form had 44% of the strength of that with bonded aggregates.
4. Specimens with untreated flat plates with T-shape ribs showed a remarkable behavior characterized by pseudo-ductility arising from gradual slip and eventually shearing off, of the rib from the base. When bonded aggregate treatment was used, concrete shear failure occurred at higher strength with no pseudo-ductility.

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Table 1 – Summary of test matrix (1 in = 25.4 mm)

Table 1. Summary of test matrix (1 m × 200 mm)											
		Panel types		Bond Mechanism			Cross sectional details				
		Corrugated	Ribbed	Nothing	Adhesive	Bonded Aggregate	Width b (mm)	Reinf. ratio %	Total depth h (mm)	Effective depth d (mm)	(a/d)
Control Specimens	S1	x		x							
	S2	x			x		270	3.5	225	198	3.8
	S3	x				x					
	S4		x	x							
	S5		x			x	290	2.34	237	228	3.3
Specimens Subjected to 300 Freeze-	S6	x			x						
	S7	x				x	270	3.5	225	198	3.8
	S8		x	x							
	S9		x			x	290	2.34	237	228	3.3
* shear span (a)=0.75 m											

Table 2 – Summary of test results (1 kip = 4.448 kN, 1 in = 25.4 mm)

	Peak load (KN)	Maximum strains		Mid span deflection at peak load (mm)	Load at 0.1 mm slip (KN)	Slip at peak load (mm)	Failure mode
		Top (micro)	Bottom (micro)				
S1	17.9	-107	61	0.47	-	0.01	Bond failure
S2	85.8	-1060	3411	10.49	50.4	0.28	Concrete shear (diagonal) - Bond failure
S3	68.1	-1013	4551	8.08	54.6	0.18	Concrete shear (diagonal) - Bond failure
S4	48.4	-219	1867	4.96	29.8	0.32	Horizontal shear of GFRP rib from base
S5	111.2	-978	3338	8.04	29.2	3.63	Concrete shear (diagonal)
S6	67.8	-890	4320	7.50	35.4	0.38	Bond failure
S7	55.6	-700	2948	7.15	34.8	0.41	Bond failure
S8	48.1	-202	2054	5.76	33.5	0.50	Horizontal shearing of GFRP rib from base
S9	112.3	-1086	3827	7.81	43.0	0.53	Concrete shear (diagonal)

Shape Effect of FRP Stay-in-Place Structural Forms on Durability under Freeze-Thaw Cycles

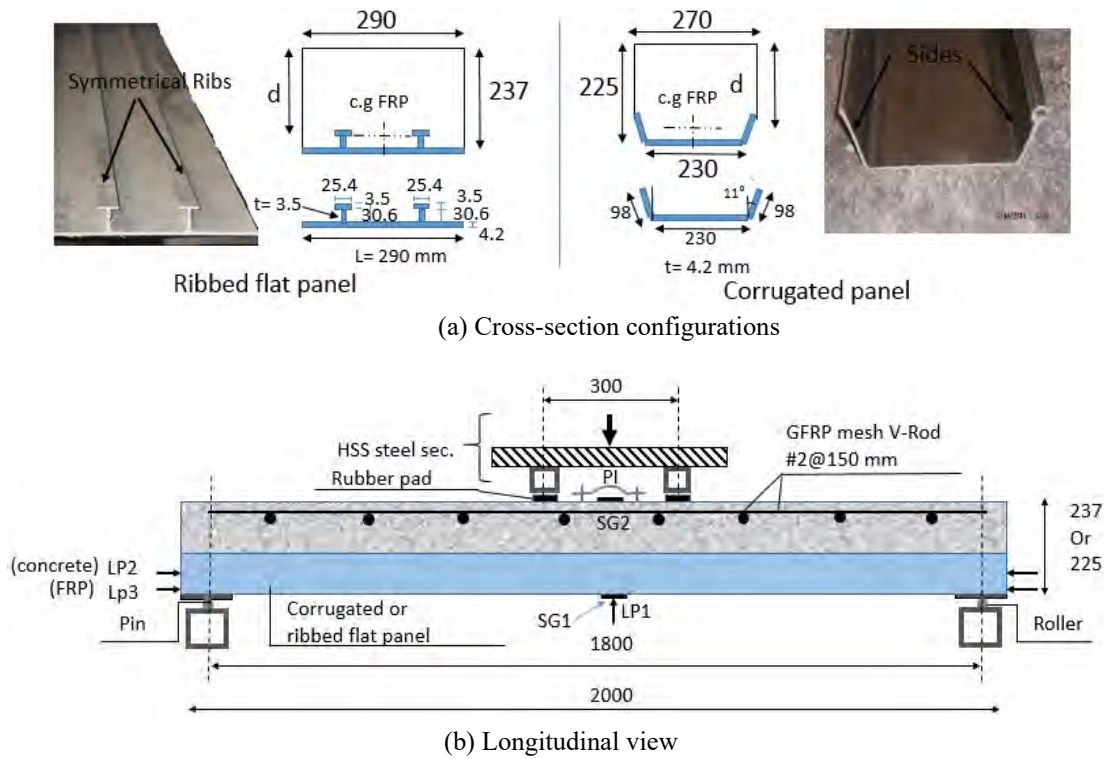


Figure 1 – Schematic diagram of test specimens, setup and instrumentation (1 in = 25.4 mm)

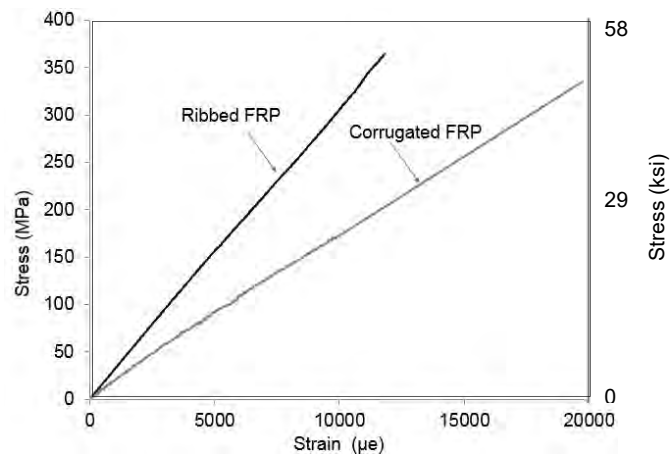


Figure 2 – GFRP SIP form stress-strain curves

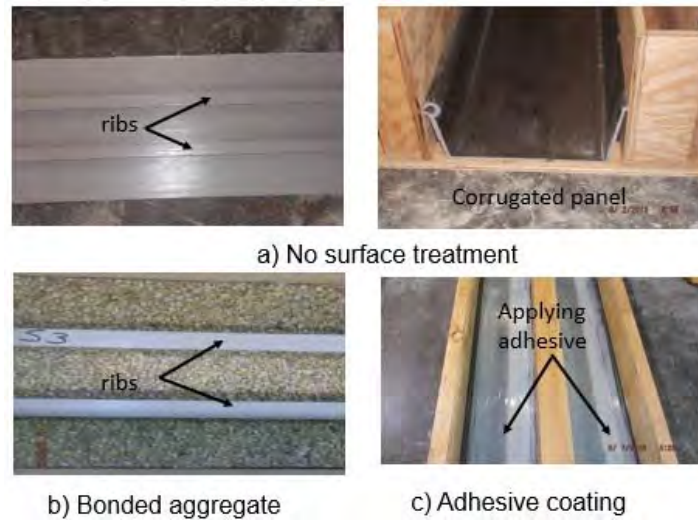


Figure 3 – Various surface preparations

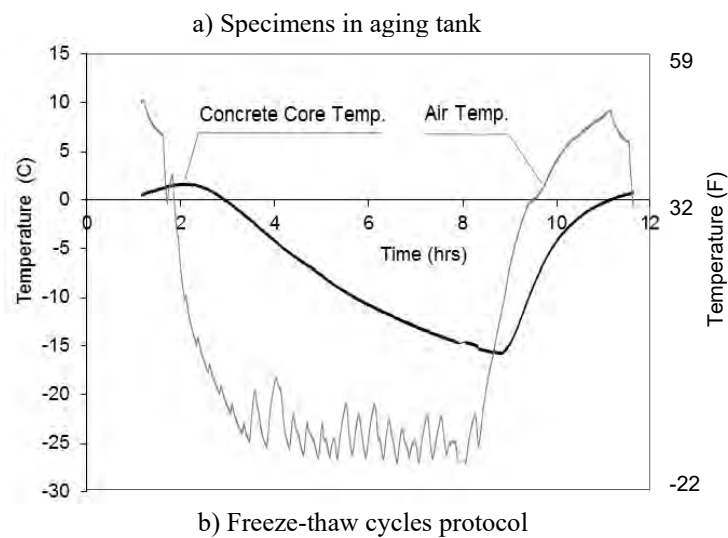


Figure 4 – Conditioning setup showing the water tank inside cold room

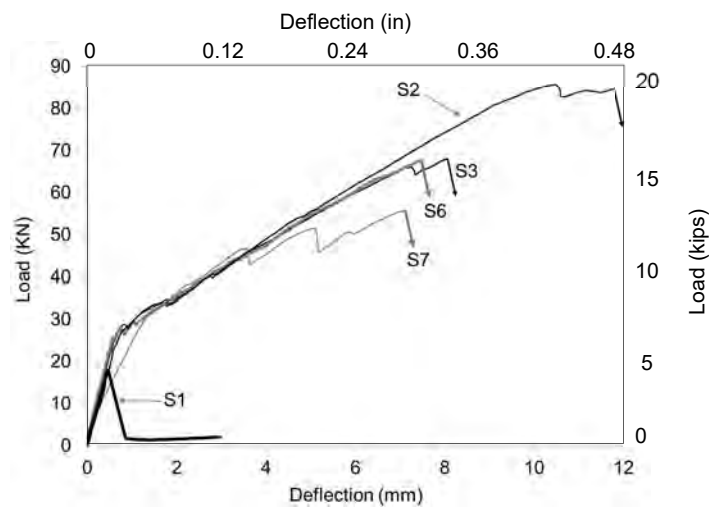


(a) Specimens with flat ribbed form

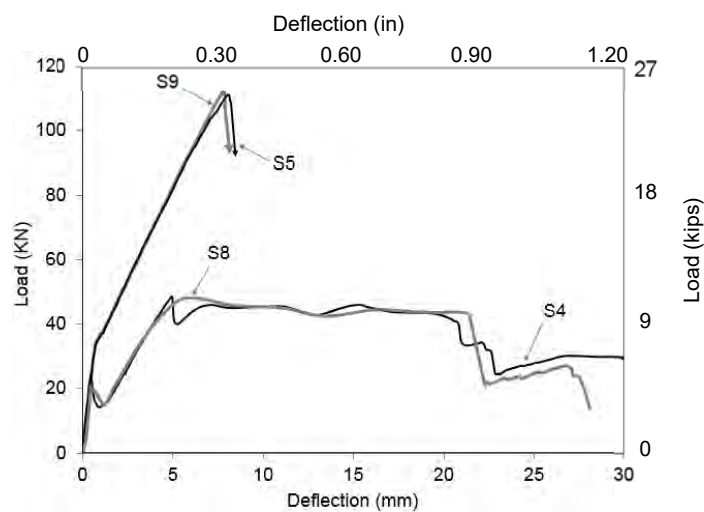


(b) Specimens with corrugated form

Figure 5 – Test setup

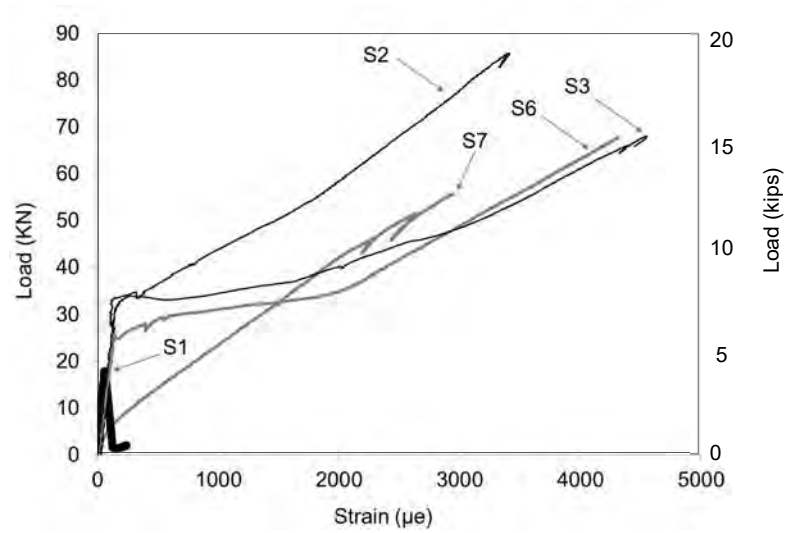


a) Corrugated panel specimens

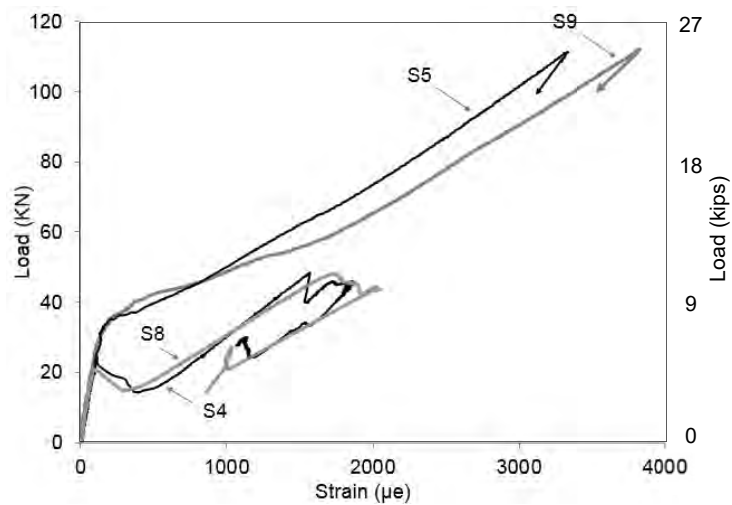


b) Ribbed panel specimens

Figure 6 – Load-deflection responses

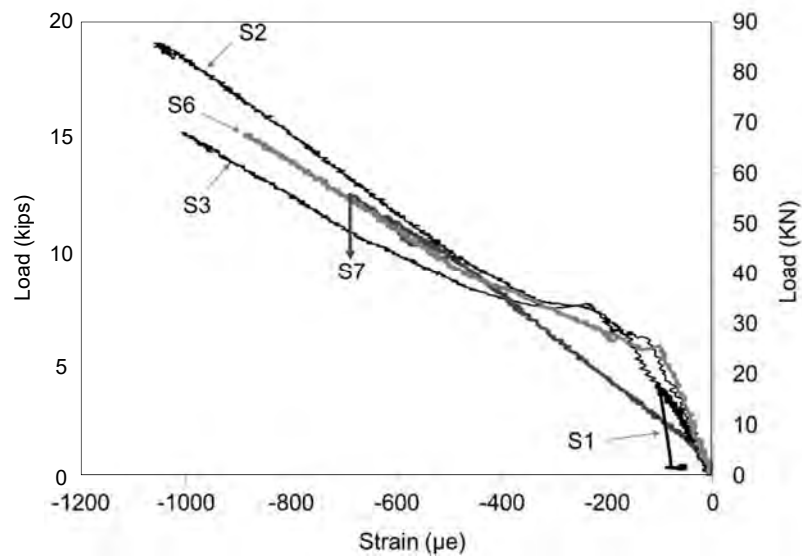


a) Corrugated panel specimens

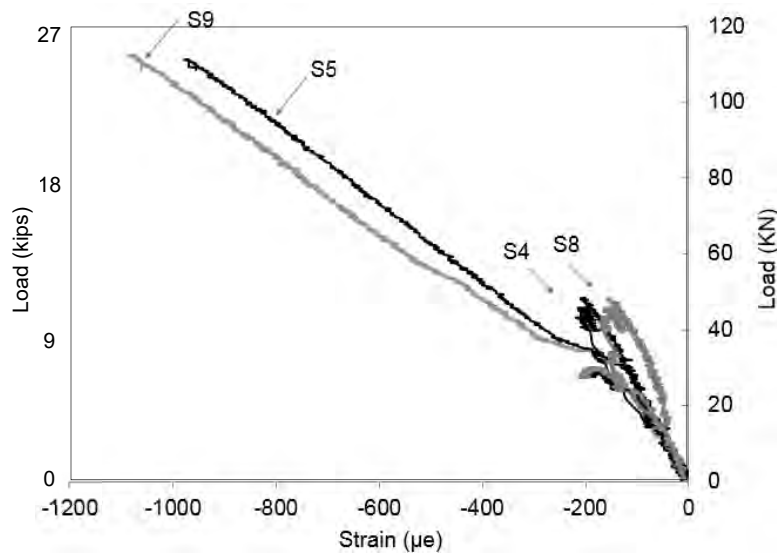


b) Ribbed panel specimens

Figure 7 – Load-FRP bottom strain responses

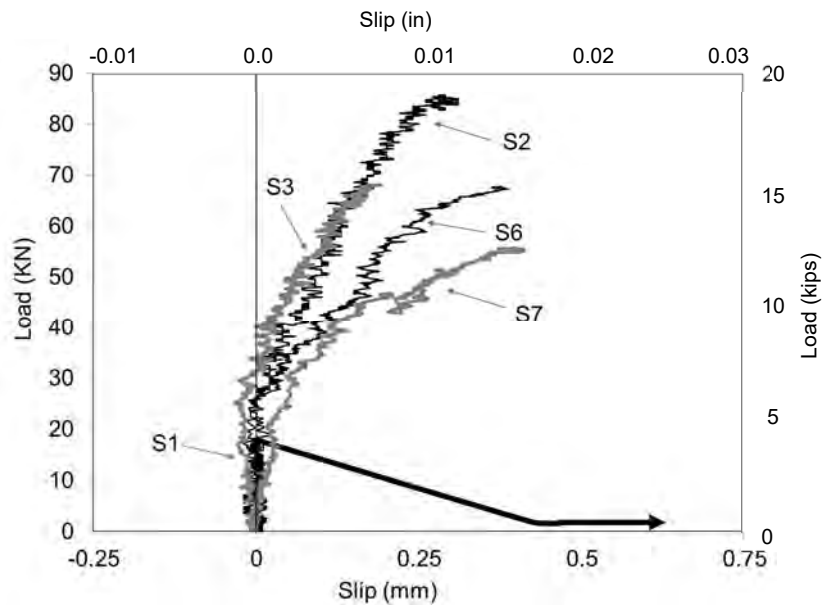


a) Corrugated panel specimens

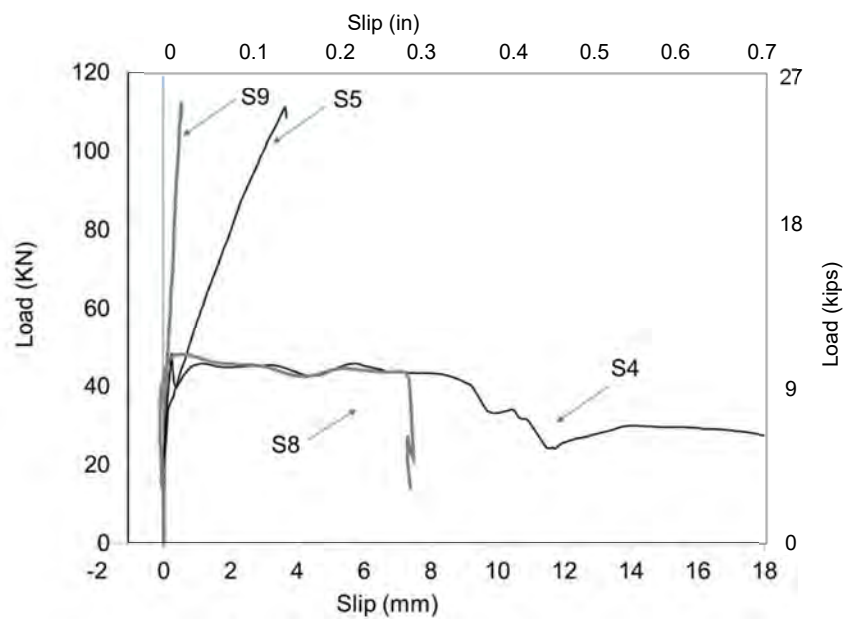


b) Ribbed panel specimens

Figure 8 – Load-concrete top surface strain responses

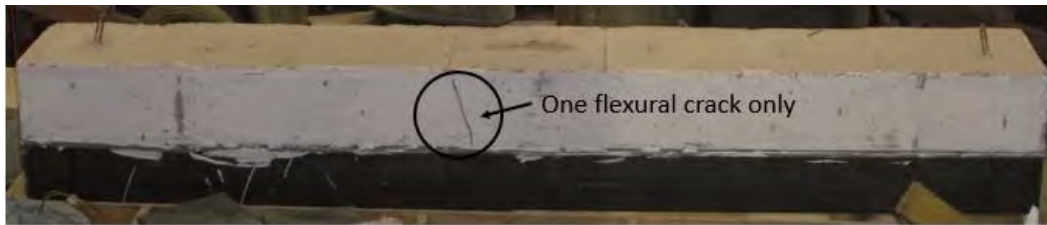


a) Corrugated panel specimens



b) Ribbed panels

Figure 9 – Load-slip responses



(a) Bond failure in S1 upon first cracking



(b) Concrete shear failure (diagonal) with bond failure in S2 and S3



(c) Horizontal shear failure of GFRP ribs from base in S4 and S8



(d) Concrete shear failure (diagonal) in S5 and S9



(e) Bond failure occurred in S6 and S7

Figure 10 – Failure modes

Shear Strengthening of Hollow Core Slab Webs using FRP Composite Sheets

Yuanli Wu, Amr El Ragaby and Shaohong Cheng

Synopsis: Precast, prestressed hollow core (PHC) slabs are among the most common concrete deck system in the world. However, due to the manufacturing constraints and the difficulty in providing internal shear reinforcement, the shear capacity of PHC slabs sometimes dictates the design and reduces the efficiency and economics of PHC slabs. The objective of this research project is to develop an innovative application of externally bonded Fiber-Reinforced Polymers (FRP) sheets by installing the sheets along the internal perimeter of the slab voids to strengthen the web-shear capacity of PHC slabs. To explore the feasibility and to optimize the new technique, experimental testing was carried out on eight full-scale single web, I shape, specimens (each of 4575 mm “180 in” long, 300 mm “12 in” thick and a 284 mm “11.2 in” wide) that were cut longitudinally out of the PHC slab. Carbon FRP sheets were bonded along the full perimeter on each side of the web specimens. The test specimens were loaded monotonically until failure under single concentrated load at a shear span/depth ratio of 2.5. The investigated parameters were the width of the FRP strengthened zone (300 “12 in.”, 450 “18 in.”, and 600 mm “24 in.”) and the number of strengthening layers (2 and 4 layers). The test results showed the efficiency of the proposed technique to enhance the shear strength of PHC slabs.

Keywords: Shear capacity, hollow core slab, web, and FRP.

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INTRODUCTION

Precast, prestressed hollow core (PHC) slabs are among the most common load bearing concrete elements in the world. They are widely used in floors and roofs of office, residential, commercial and industrial buildings. Their applications in wall panels and bridge deck units are also very popular. With tubular voids extended over the entire slab, the self-weight and fabrication cost of PHC slabs can be considerably reduced. Compared to an ordinary reinforced concrete slab, a PHC slab with the same resistance capacity can save 40% to 50% of steel and 20% to 40% of concrete [1]. Its reduced thickness would help to conserve headroom, which is particularly important for multi-story buildings. Figure 1 shows the cross-section of a typical PHC slab and an I-shaped single web beam cut from the slab. The latter can also be considered as a single web hollow core slab. Prestressed strands are provided to the PHC slabs not only to increase their strength but also reduce deformation under normal conditions.

Although PHC slabs are generally designed to resist flexural stress induced by assumed uniformly distributed loads, in practice, they could also be subjected to concentrated loads or line loads which would possibly lead to shear failure, in particular, over regions close to the end support. However, the dry cast or extrusion technique used to manufacture PHC slabs makes it difficult to provide shear reinforcement during manufacturing processes. Thus, PHC slabs typically rely on the shear strength of plain concrete to resist shear stresses. When the shear capacity of the PHC slabs is exceeded, the common practice is to either fill the voids with additional concrete or use thicker slabs to satisfy the shear strength requirement. However, both solutions would add cost, increase self-weight and thus reduce the economy of PHC slabs. Therefore, there is an urgent need to develop an efficient technique to improve the shear capacity of PHC slabs while still retain its advantages.

One of the typical shear failure modes of PHC slabs is web shear failure. It occurs when the maximum principle stress resulted from the combined effect of shear and bending exceeds the tensile strength of concrete. As a consequence, a crack would initiate from a point on the critical section. Subsequently, more and more cracks would form and propagate upwards and downwards in a straight inclined form joining the edge of the slab support with the top of the slab and eventually lead to total collapse of the slab. The failure is mainly induced by diagonal cracks formed in the PHC slab web at the critical section near the support. Therefore, if the shear capacity of the PHC slab webs can be enhanced, the shear capacity of the entire PHC slab can be increased as well [1,2, 3].

According to existing studies [1, 2], there are a number of factors that could influence the shear capacity of PHC slabs. Among these, the most important ones are identified to be: a) Size of the cross section: the depth of slab can directly affect its ultimate shearing strength, just as beams [3]; b) Level of prestressing: number and distribution of prestressed reinforcement would alter the shear capacity of PHC slabs [5, 6]; c) Concrete properties: this is an important parameter since it will determine the compressive and tensile strengths [1,3]; d) Shear span/depth ratio: this parameter would also affect the ultimate shear capacity of PHC slabs. The critical shear span/depth ratio was found to range between 2 and 3 [6]; and e) The interaction between adjacent webs on the shear strength of prestressed concrete hollow-core units which may lead up to 20% increase in the shear capacity [4].

In recent years, externally bonded fiber reinforced polymer (FRP) composite sheets have been successfully used in the repair and rehabilitation of reinforced concrete beams and slabs. The feasibility of using FRP composites to

strengthen shear capacity of reinforced concrete beams was first explored by Berset [8]. Glass fiber sheets were used to strengthen the lateral shear zone of reinforced concrete beams. By comparing the behaviour and capacity with the control beam, a shear capacity analysis model for the FRP strengthened beams was proposed. The contribution of FRP sheets to the shear resistance capacity of the beam was found to be based on the maximum allowable strain. The effectiveness of this shear strengthening technique was confirmed by Chajes et al. [9] and Triantafilou [10], of which the former series of tests showed a 50% to 60% increase in shear capacity, while the latter indicated a FRP orientation of 45° and 135° with beam axis would be more effective than the 90° arrangement. Subsequently, Sundarraja and Rajamohan [11, 12] provided another set of experimental data from a shear strengthening test on reinforced concrete beams externally bonded by bi-directional FRP strips, in the form of either inclined strips or U-wrap strips. Results indicated that FRP strips were effective in shear strengthening of structures since the initial cracks appeared at higher loads than their respective control beams. Also, the ultimate strength of beams could be increased when inclined or U-wrap FRP strips were bonded, with the latter being more effective. The width of the FRP strips was also found to be an important parameter determining the shear strengthening effect. Panigrahi et al. [13] studied shear strengthening of reinforced concrete T-beams using epoxy bonded bi-directional GFRP sheets. The investigated parameters included the amount and distribution of GFRP, the bonding surface, the number of layers and the orientation of GFRP fibre. Results confirmed that the externally bonded GFRP technique could be used to enhance the shear capacity of reinforced concrete T-beams.

Based on the promising results of the above studies, the authors are keen to implement a similar technique to improve the shear capacity of PHC slabs. It is proposed to bond carbon FRP sheets along the full perimeter of PHC slab voids to enhance its shear strength. Extensive experimental testing and finite element modelling have been carried out at The University of Windsor to investigate the effect of FRP sheets bonded along the full perimeter of slab voids on the performance of PHC slabs. The investigated parameters included the depth of PHC slabs, the prestressing level (number of prestressing strands in one web), the type and thickness (number of layers) of FRP sheets as well as the width of the strengthened zone and the shear span/depth ratio. To explore the feasibility and effectiveness of this new technique in terms of the enhancement of ultimate shear capacity and to determine the effect of different test parameters, the technique was applied first to I-shaped single web beams cut longitudinally from the full PHC slabs, which can be considered as single web hollow core slabs. This paper focuses on the experimental results of full scale I-shaped single web beams, which are longitudinally cut out from PHC slabs and externally strengthened by Carbon FRP (CFRP) sheets bonded along the full perimeter on both sides of the specimen, as shown in Figure 2. The studied parameters included the width of the strengthened zone and the thickness, the number of layers, of CFRP sheets.

RESEARCH SIGNIFICANCE

Precast, prestressed hollow core (PHC) slabs are among the most common load bearing concrete elements in the world. However, due to lack of internal shear reinforcements and the difficulty in providing internal shear reinforcement within the webs due to the nature of the manufacturing process, the shear capacity under concentrate or line loads near the support is a serious problem which in many cases dictate the design and reduces the efficiency and economics of PHC slabs. This research is dedicated to develop an efficient and feasible technique to strengthen the shear capacity of PHC slabs while still retain its advantages. Experimental testing and finite element modeling have been carried out to investigate the efficiency of strengthening the shear capacity of PHC slabs using FRP sheets externally bonded on the PHC slab webs, along the internal full perimeter of the slab voids.

EXPERIMENT PROGRAM

Specimens

A total of eight I-shaped single web beams, cut from full PHC slabs, were used in the experimental study. All the specimens had a length of 4575 mm (180 in), a depth of 300 mm (12 in) and a width of 284 mm (11.2 in), which represents the centre to centre distance between two adjacent webs in a full PHC slab. Each specimen had two longitudinal prestressed steel strands, each of 13 mm (0.5 in) diameter, at the bottom. All specimens were tested over a simply supported clear span of 4499 mm (177.12 in.) while the bearing width at each support is 76 mm (3 in.). The support condition is a pin at the loading end and a roller on the other end, so that no axial forces are generated by rotation or the constraint of the web beams at the support. One concentrated load was applied monotonically till failure

at a distance of 750 mm (29.5 in) from the pin support via a very stiff steel plate of 190 mm width. This results in a shear span to depth ratio (a/d) of 2.5. The details of the experimental test setup is shown in Figure 3.

Among the eight specimens, two of them are used as control beams, (W2-C1 and W2-C2), for which no FRP sheets were applied, where “W” represents web beam specimen, “2” represents two prestressed steel strands, and “C” represents control beam. Three different widths of the strengthened zone of 300 mm (12 in), 450 mm (18 in) and 600 mm (24 in) were investigated. In addition, two different thicknesses of FRP strengthening, two layers FRP sheets (one layer per side) and four FRP layers (two layers per side) were investigated. The ID of the rest six specimens is defined to include these characteristics of FRP sheets with a pattern of W2-number of FRP sheets-length of FRP sheets. For example, in specimen W2-4-300, four FRP sheets (two on each side) of 300 mm (12 in) length are externally bonded to the web beam specimen that has 2 prestressing strands. Table 1 lists the details of each specimen.

Instrumentations

Each PHC web specimen was instrumented at the loaded end. As shown in Figure 4(a), two linear variable displacement transducers (LVDT) were installed, one at the loaded end on one prestressing strand to measure its end slippage during tests and another at the top flange of the beam specimen adjacent to the load to measure vertical displacement. In addition, four displacement transducers, Pi-gauges, were used to measure strain at different locations. One is placed on the surface of the specimen top flange in the longitudinal direction near the loading position to measure the top concrete compressive strain while the other three transducers were located at the intersection points between the web longitudinal centre line and the 35° line from the support plate, the interior line from the support plate to the loading plate and a line connecting a point at a distance of $d_v=225$ mm from the interior edge of the support plate at the bottom surface to the middle of the loading plate, respectively, as shown in Figure 4(b). These three critical points were calculated according to the critical shear crack position based on EN 1168 [14], previous literature [6] and CAN/CSA-A23.3-04 code [15]. In addition, three electrical foil strain gauges were glued on the face of the FRP sheets, in the vertical direction at the center point of each Pi-gauge to monitor FRP tensile strain.

Material properties

The test specimens were constructed using the same batch of normal-weight concrete with an average 28-day compressive strength of 60 MPa (8702 psi). 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 as their reference slabs to evaluate the compressive and tensile strength of concrete at the test time. SikaWrap-900C CFRP sheets and Sikadur300 Epoxy were used in strengthening the PHC web specimens. The SikaWrap-900C is a uni-directional, fleece stabilized, stitched heavy carbon fiber fabric used for wet-lay-up application of structural strengthening [16]. The mechanical properties of the FRP sheets with epoxy resin, provided by the supplier and determined by tensile tests on representative samples in accordance with ASTM D3039 [17], showed a linear-elastic stress-strain relationship with average values of elastic modulus and tensile strength of 100 GPa (145×10^5 psi) and 1120 MPa (162×10^3 psi), respectively. The prestressing strands are 7-wire, low-relaxation strands which have a diameter of 13 mm (0.5 in) and ultimate tensile strength of 1860 MPa (270×10^3 psi).

Preparation of test specimens

The specimens were fabricated, cured and cut at the supplier site, then were shipped to the Structures lab at the University of Windsor for testing. The specimen surface where the FRP sheets would be applied was cleaned and polished using a steel brush drum derived by a drill, in the same way as the full PHC slab specimen were cleaned, before attaching the FRP sheets. The uni-directional FRP sheets were applied in the vertical direction to the web surface using a wet layup process. Both the CFRP sheets and the concrete surface were first saturated by epoxy resin, then the sheets were directly bonded to the specimen surface, as shown in Figure 5. The direction of FRP fibres were oriented in the direction of the web depth, perpendicular to the longitudinal axis of beams. They were left for curing for 7 days indoor before testing.

Testing procedures

Following the Deutsche Norm. (2008) [14], the load was applied in two steps. The specimens were first loaded to 70% of the estimated failure load at a rate of 10 kN (2.25 kip) per minute then unloaded. In the second load cycle, the

load was applied monotonically at the same rate until failure. According to the ultimate load of the two control specimens, W2-1C and W2-2C, (both failed at ultimate load of 81.5 kN (18.3 kip) and 76.3 (17.2 kip), respectively), the peak load in the first load cycles was chosen as 56 kN (12.6 kip). Figure 6 shows a typical specimen during loading stage.

EXPERIMENTAL TEST RESULT AND OBSERVATIONS

Cracking behaviour, and failure mode

The two control specimens showed almost identical behaviour. At 30 kN (6.7 kip), the first crack appeared at the bottom surface near the loading end at the face of the bearing plate. Close to failure load, an inclined crack initiated at the support and propagated quickly through the beam web up to the top flange close to the loading plate, over the entire shear-tension region with a distance of 780 mm (30.7 in) from the support. The maximum width of this critical shear crack was 3.2 mm (0.1 in). There was no formation of vertical flexural crack prior to beam failure. Both W2-1C and W2-2C failed suddenly in diagonal shear at ultimate load of 81.5 kN (18.3 kip) and 76.3 (17.2 kip), respectively. An average value of 78.9 kN for the ultimate shear capacity of the control specimens will be considered in the comparisons. Figure 7(a) shows the typical cracking and failure shape of the control specimens.

Figure 7(b) shows a typical cracking pattern and failure shape of W2-2-Series specimens. The first vertical crack of W2-2-450 web beam appeared at the bottom of the flange at loading end when load was 13 kN (2.9 kip). When the load reached 79.9 kN (17.9 kip), a critical inclined crack initiated at the inner edge of the support bearing plate at the loading end and grew upwards till reached top flange of the specimen at the inner edge of the loading plate. After failure, the FRP sheets were mechanically peeled off the concrete surface to measure the shear crack location and width. The horizontal distance between the crack top end and the beam loading end was measured to be 715 mm (28 in), and the maximum crack width was 2.0 mm (0.1 in). No formation of vertical flexural crack was observed before beam failure. The failure mode of this specimen belonged to shear failure, too.

W2-4-series specimens showed different cracking behaviour before failure. The cracking pattern and failure mode of W2-4-300 web beam is shown in Figure 7(c). The first vertical crack appeared at the bottom of the loading end when the load reached only 10 kN (2.2 kip). Furthermore, in all three cases, vertical flexural crack(s) formed at the bottom of the beam close to mid-span prior to beam failure. In the case of W2-4-300, one such crack appeared at around 90 kN (20.2 kip), whereas two vertical flexural cracks were observed in W2-4-450 and W2-4-600, when the load reached respectively around 80 kN (18 kip) and 100 kN (22.5 kip). This results in more ductile behaviour before failure. The critical inclined crack initiated at the specimen bottom at 174 mm (6.9 in) from the pin support at the loading end and extended to the top flange close to the middle of the loading plate. The recorded ultimate shear failure load was 102.1 kN (30 kip). Further, the critical shear cracks in both W2-2-450 and W2-4-300 web beams were observed to grow through the beam web. In all the strengthened specimens, no de-bonding or delamination of the FRP sheets was observed at all before failure. FRP sheets were de-attached manually from the concrete surface to study the main inclined shear crack.

In addition, for all eight tested web beam specimens, the critical shear failure cracks were found to break through the top flange of the specimens in the compression zone, as can be seen in Figures 8(a), 8(b) and 8(c). Also, during the tests, formation of vertical and radial cracks was observed around the prestressing strand at the loaded end before the load reached 35 kN (7.9 kip). Figures 9(d), 9(e) and 9(f) show some sample cases. The occurrence of these early vertical cracks were mainly due to the high prestressing force in the steel strands. So the surrounding concrete, which was much weaker, was easier to damage at the loading end. This would reduce the specimen stiffness at the very beginning of the first loading stage.

Enhancement of the ultimate shear capacity

The testing results of the 8 web beam specimens are summarized in Table 2. The average shear failure load of W2-C control web beams is 78.9 (17.7 kip). The failure load of W2-2-300, W2-2-450 and W2-2-600 are 76.8 kN (17.3 kip), 77.9 kN (16.2 kip) and 83.5 kN (18.8 kip), respectively, which are almost the same as the control specimen, i.e. no improvement on shear capacity. As discussed in the previous section, the cracking behaviour of W2-2 series was also very similar to the control specimen. This implies that using two layers of FRP sheets to strengthen the shear

capacity of the studied web beam is not adequate. In addition to acting as external stirrups to enhance the shear capacity, the CFRP sheets are also expected to provide more confinement for the concrete in the tension zone around the strand and thus reduce the strand slippage. As some radial and vertical cracks were already observed around the strand at the loading end during testing, installing only 2 layers of CFRP sheets seems to be inadequate to provide sufficient confinement around the strand especially close to the support and thus failed to enhance the ultimate capacity. More tests are needed to confirm this finding. The failure mode of the W2-2-Series specimens remains the same as that of the control beam, i.e. failed in brittle shear failure mode. However, shear capacity of the W2-4-Series webs were found to be greatly enhanced with the application of the proposed shear strengthening technique. Compared to the shear failure load of the control beam, the ultimate shear failure load of W2-4-300, W2-4-450 and W2-4-600 are 102.1 kN (23 kip), 99.6 kN (22.4 kip) and 114.2 kN (25.7 kip) respectively. It can be concluded that the shear capacity of PHC webs increased by 29.4%, 26.2% and 44.7% when 4 layers of CFRP sheets were bonded to the web surface for a length of 300, 450 and 600 mm respectively. These results imply that by increasing the thickness, number of layers, of FRP sheets and the width of the strengthened zone, shear capacity of the studied web beams can be considerably increased.

Load deflection behaviour and ductility

Figure 9(a) shows the load-displacement relationships of the control beam and the three W2-2 series specimens. The trend and pattern of the four curves are found to be very similar. Besides, the maximum displacements of the four beams at the loading position are almost the same, which are about 4-5 mm (0.2 in) which indicates sudden and brittle shear failure occurred in these four specimens. On the other hand, in the case of the W2-4-series specimens, the maximum displacement at the loading point are much larger. In particular, as can be seen from Figure 9(b), it exceeded 10 mm (0.4 in) in W2-4-450 and W2-4-600. This is more than two times of that of the control beams and clearly shows the improvement in the ductility of the W2-4 series specimen with 2 layers of FRP sheets bonded at each side of the web. Further, although W2-4-600 specimens exhibit the best shear strengthening effects in terms of enhancing shear capacity and ductility, considerable improvement was also achieved in W2-4-300 and W2-4-450 specimens. Compared to 600 mm (24 in) wide, CFRP strengthened zones with widths of 300 mm (12 in) and 450 mm (18 in) would be easier to install and make the application of the proposed shear strengthening technique more feasible in real application.

Figure 10 shows the relationship between the load and end slip of the prestressing strands at the loaded end. It can be noticed that specimen W2-4-450 suffered from significant strand slippage at the loading end mainly due to the formation of early cracks on the loaded face around the strands. This also explains the low stiffness W2-4-450 specimen shown in Figure (9-b)

Critical shear crack profiles

Figure 11 shows the crack profile of each specimen at failure. As can be observed, in all cases, a diagonal shear crack formed at the instant of failure, indicating a typical shear failure. The orientation of the cracks is between 17°-46° with respect to the specimen longitudinal axis. A closer look at this set of results reveals that in the case of the control beam, the critical shear crack initiating point is very close to the intersection of the specimen longitudinal axis and the interior line connecting the inner edges of support and loading bearing plates, which is consistent with existing studies [5, 6]. For W2-2series specimens the diagonal shear crack and the effective shear-tension region is very similar to the control specimen. The diagonal shear crack extended from the support at the bottom face toward the loading plate at the top face of all W2-2 series specimens. This might explain the inadequacy of two layers of FRP sheets in strengthening the shear capacity of PHC. However, it is interesting to note that for all three W2-4 series specimens, the shear-tension region is found to be shifted away from the support. This could be mainly due to the presence of four FRP sheets close to the specimen support which considerably enhances the shear resistance capacity in this local region. The fact that four FRP sheets showed much strong enhancement of shear capacity than the two FRP sheets arrangement seems to suggest that there could exist a minimum requirement of FRP sheets to obtain satisfactory shear strengthening effect. More tests are needed to clarify this.

Strain behaviour

Figure 12 shows load vs top longitudinal concrete compressive strain relationships measured on the top flange at 565 mm from the loading end. It can be noticed that for W2-2-series, the measured values of the top concrete