DEVELOPMENT OF A COMPATIBILITY MODEL FOR THE SHEAR STRENGTHENING OF REINFORCED CONCRETE CONTINUOUS BEAMS USING EXTERNALLY BONDED BI-DIRECTIONAL CFRP STRIPS

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ABSTRACT

The behaviour of reinforced concrete in shear is very complex and structure that fails in shear was dangerous than flexural failure because shear failure usually occurs suddenly and without any warning. It has been found out that many of existing reinforced concrete structures are deficient in shear and needs to be repaired. Therefore, this study tends to investigate the shear behaviour of reinforced concrete beams strengthened and repaired using Carbon Fibre Reinforced Polymer (CFRP) composites. An experimental work on ten reinforced concrete continuous beams has been conducted. The parameter involved in this study was shear span to effective depth ratio, wrapping schemes and CFRP strips orientation. All beams fails in shear as expected. Modes of failure for beams wrapped at three sides of the beam were debonding and rupture of the CFRP strips while beams wrapped at four sides of the beam fails in rupture of the CFRP strips. The highest shear capacity enhancement was for beam wrapped at four sides of the beam with 45°/135° orientation of the CFRP strips. From the experimental results, it can be concluded that wrapping schemes influences the shear capacity and diagonal crack angle of the beam. It was also important to notice that for continuous beams, due to the complexity of the high shear and bending moment region, the critical area (inner shear span) of the high shear area should be strengthened more than the outer shear span of the beam.
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CHAPTER 1

INTRODUCTION

1.1 Introduction

In this few decades, the use of Fibre Reinforced Polymer (FRP) as a strengthening material is a well known method in civil engineering. However, prior to civil engineering, in the early days back to 1930s, FRP materials have been widely used in many areas such as aerospace, transportation, maritime and electrical (ACI Committee 440, 2006). In civil engineering, old building such as historical building that needs to be taken care of besides the defects on newly built reinforced concrete structures are among the reasons why FRP has been used to strengthened and repair the structures. The most important reason why FRP was chosen are because of its characteristics such as high resistance to corrosion, high strength to weight ratio, ease of installation, nonmagnetic, resistant to chemicals and high tensile strength. These advantages are among the reasons of the use of FRP materials as an alternative solution to strengthen existing structures (Adhikary and Mutsuyoshi 2004; Chajes et. al 1995; Khalifa et. al 1998; Khalifa and Nanni 2000; Grace et. al 1998; Norris et. al 1997; Taljsten 2003). The old building and structures has motivated many researchers and organization to find alternative materials and technique to restore deteriorating and deficient structures (Taljsten, 2003). In Malaysia for example, the use of FRP as a strengthening and repair material was implemented at the Middle Ring Road 2 (MRR2) in Kepong where the flyover was seriously damaged and cracks was clearly seen at the piers and girders. To overcome the problem, FRP was used to strengthen the structure.
1.2 Problem Statement

Structure that fails in shear was dangerous than flexural failure because shear failure usually occurs suddenly and without any warning (Zhang and Hsu, 2005, Jayaprakash et al, 2008 and Khalifa and Nanni, 2000). In general, shear failure is a diagonal tension failure that is brittle in nature and should be avoided (Wang, Salmon & Pinheiro, 2007). The behaviour of reinforced concrete in shear is very complex as the current code and design procedures are based on analysis of experimental results and model assumption rather than on an exact universally acceptable theory. The complexity are due to the non-homogeneity of material, nonlinearity of material, cracks, presence of reinforcement, load effects and many other things (Pillai, Kirk & Erki, 1999).

Therefore, it is important to strengthen a structure that is deficient in shear where many of existing reinforced concrete structures has been found to be deficient in shear and need to be repaired (Khalifa and Nanni, 2000). There are several reasons of the deficiencies such as insufficient shear reinforcement, increase of service load and corrosion of the reinforcement. Generally, most of investigations which were carried out focused on shear strengthening of reinforced concrete simply supported beam. However, the existing model has not been confirmed for shear strengthening in areas subjected to combine high flexural and shear stresses or in region of negative moment (ACI Committee, 2008), whereas most of existing beam was in the form of continuous condition. Furthermore, there are restraints to add shear reinforcement to an existing reinforced concrete beams when beams are part of floor-beam system.

1.3 Objective

The main objective of this study is to investigate the effectiveness of using externally bonded bi-directional Carbon Fibre Reinforced Polymer (CFRP) composites as a strengthening method for continuous beams. In specific, the objectives of this study are:
a) To conduct an experimental investigation on strengthening and repair of reinforced concrete continuous beams using CFRP strips due to shear failure at $a_t/d=2.5$ and 3.5.

b) To verify the experimental investigation using computational finite element modeling and existing theoretical formula.

1.4 **Scope of Study**

This study focuses on shear strengthening and repair of reinforced concrete continuous beam using externally bonded CFRP sheet. Therefore, in order to achieve the above-mentioned objectives, this study was conducted within certain scopes and limits as follows:

i) This study involves an experimental work on two-span continuous beams with a size of 150mm width, 350mm depth and 5800mm of total length.

ii) All of the specimens were designed to fail in shear and were provided with sufficient flexural reinforcement so that it will not fail in flexure.

iii) The specimens were divided into two groups of different shear span to effective depth ratio, i.e. 2.5 and 3.5 respectively.

iv) All of the specimens have identical design detail and the same concrete compressive strength of $30N/mm^2$.

v) The bi-directional CFRP sheet was selected for the experimental program.

vi) The width of the CFRP strips was selected at 80mm, whilst the spacing between strips are located at 150mm centre to centre.

1.5 **Research Significance**

Pre-existing researches have been done by many researchers on the shear strengthening of reinforced concrete simply supported beam using CFRP composites. However, strengthening continuous beams in shear have received very little attention among the researchers. Therefore this study tends to investigate the shear strengthening of
reinforced concrete continuous beams externally bonded by CFRP composites. Apart from that this study was using a bi-directional CFRP composite where this type of fibre orientation was still not widely used in most of the previous researches.

By using CFRP sheets as a strengthen material in shear, it is more practical because it is more difficult to strengthen an existing concrete structure in shear than to strengthen it in flexure.
CHAPTER 2

LITERATURE REVIEW

2.1 Advantages of FRP Composites

As mentioned earlier, FRP composites have many advantages such as good corrosion resistance, light weight, high strength to weight ratio, easy to install, non conductive and resistance to the chemicals (Triantafillou & Antonopoulus, 2000). Apart from the use of FRP in the defense area, FRP composites provide many advantages in the civil engineering arena also. With the civil engineering field that faced serious problem in many aspects such as the service life of the structures, corrosion of reinforcement, design faults and improper planning, FRP is believed as one of the solution to the problem.

The bonding of FRP plate on the tensile face has been proved to be an effective method to increase both the strength and stiffness of concrete members (Leung & Pan, 2005). Depending on the product and application, FRP products for civil infrastructure or construction applications can be optimized for specific load. Besides that, reduced dead load of FRP components of rehabilitated structures can result in increased load ratings. FRP also reduced maintenance costs because of resistance to deicing salts and other corrosive agents (ACI Committee 440, 2006). The other advantages of FRP composites are reduces field installation time by using engineered system packaging, reduces traffic delays because of the faster construction, increase reliability by pre-engineered systems, enhanced the durability and fatigue characteristics and increase the service life of the structure. Besides that, products and system enable value engineering that result in innovative and efficient installations (ACI Committee 440, 2006).
2.2 Disadvantages of FRP Composites

Besides its advantages that give promising results from the uses of the materials, FRP composites have its own drawbacks. Taljsten & Blanksvard (2007) mentioned some of the drawbacks such as the long-term properties where there were not enough data to verify the fact that FRP have a very good long-term properties as the application of FRP in the building industries have been used for only around 10 years. Apart from that, the working environment while handling the FRP composites was also important where any mishandled may cause injuries to the workers. Some FRP materials also depend on moisture and temperature for the hardening process and therefore for some environments, it was necessary to place heat to the structure.

In addition, FRP composites have low ductility where the stress-strain relationship is linear and therefore results in a sudden and brittle failure. Apart from that, when compared with steel, FRP have greater deformation than steel at the same stress level (Kodur & Baingo, 1998).

2.3 Fibre Reinforced Polymer Constituents

Fibre Reinforced Polymer is a composite material, tailored by large numbers of thin high strength fibres embedded in a plastic resin (ACI Committee 440, 2006). FRP are defined as a polymer matrix, either thermoset or thermoplastic, that is reinforced with a fiber or other reinforcing material with a sufficient aspect ratio (length-to-thickness) to provide a discernable reinforcing function in one or more directions (ACI Committee 440, 2006). Cusson & Xi (2002) states that a fiber-reinforced polymer or FRP is an advanced composite or materials system. It is defined as a solid material which is composed of two or more substances having different physical characteristics in which each substance retains its identity while contributing desirable properties. It is also a structural material made of plastic within which a fibrous material is embedded; the components remain physically identifiable exhibiting an interface between one another.
Fibres are characterized by excellent tensile strength in the direction of the fibres and negligible strength in the transverse direction. These fibres are named as unidirectional fibre system. The woven or bi-directional fabrics are made up of fibres oriented at both 0 degree and 90 degree with an equal distribution of fibres in each direction. There are many types of FRP composites based on the modifiers used in the composites such as Glass Fibre Reinforced Polymer (GFRP), Carbon Fibre Reinforced Polymer (CFRP), Aramid Fibre Reinforced Polymer (AFRP) and Hybrid Fibre Reinforced Polymer (HFRP) for composites containing different types of fibres (ACI Committee 440, 2006). The major differences between the FRP composite reinforcement and steel reinforcement are that the FRP has higher elastic strength, lower stiffness and elastic behaviour up to failure without any yielding plateau.

The performance of any composite depends on the materials of which the composite is made, the arrangement of the primary load-bearing portion of the composite (reinforcing fibres) and the interaction between the materials (fibres and matrix) (ACI Committee 440, 2006). In general, FRP composite is made by the combination of fibres and polymers: resin where the polymer resin surrounds and binds the fibres together. The resin influences the physical properties of the end product while the fibres provide the mechanical strength. Fillers and additives are used to impart special properties to the end product.

2.3.1 Fibre Reinforcement

In civil engineering, there are three types of fibres that commonly used which are glass, carbon and aramid. The fibre component consists of fine thread-like natural or synthetic material characterized by its aspect ratio (fibre length divided by fibre diameter), width length nearly 100 times its diameter (Cusson & Xi, 2002).

Glass fibres are produced by extruding molten mass through an orifice of 0.79-3.18 mm in diameter followed by drawing through fine opening of 3-20 μm in diameter (ACI Committee 440, 2006). Glass fibres are commercially available in E-glass formulation (for electrical grade), the most widely used general purpose form of composite reinforcement and other formulations for high strength (S-2 glass), improved
acid resistance (ECR glass) and alkali resistance (AR glass) (ACI Committee 440, 2006). The advantage of glass than carbon and aramid is it is a good impact-resistant fibre. However, glass is denser than carbon and aramid. The end product of glass fibre is they are very good electrical and thermal insulation materials. Apart from that, glass fibre also used in radar antenna applications due to its characteristics which is transparent to radio frequency (ACI Committee 440, 2006).

Carbon fibres are also known as graphite fibres. Three natural resources supply the production of structural carbon fibres: pitch, a product of petroleum distillation; PAN, polyacrylonitrile and rayon (Cusson & Xi, 2002). The properties of carbon fibres are controlled by molecular structure and degree of freedom from defects. The formation of carbon fibres requires processing temperatures above 1000°C (1830°F) (ACI Committee 440, 2006). There are two types of carbon fibres which are high modulus and high strength. The advantages of carbon fibres are high strength, high strength to weight ratio, low longitudinal and transverse coefficient of thermal expansion, low sensitivity to fatigue loads and excellent moisture and chemical resistance. Carbon fibre is about five to ten times more expensive than glass fibre, however, it has about twice the usable strength and four times the modulus of glass (ACI Committee 440, 2006).

The aramid or aromatic polyamide fibres are manufactured by extruding polymer solution through a spinneret resulting in a fibre with high thermal stability, high strength and high stiffness. The aramid fibres are fire resistant and perform well at high temperatures. The tensile strength of aramid fibres is higher than that of glass fibres and the modulus is about 50% higher than that of glass (ACI Committee 440, 2006). The other advantages of aramid fibres are resistant to organic solvents, fuels and lubricant, increase the impact resistance of composites and they also a good insulator of electricity and heat. However, aramid composites have poor compressive strength and composites using such aramid fibres should be carefully designed, especially for compression or bending (ACI Committee 440, 2006). Figure 2.1 shows the stress-strain relationship between CFRP, GFRP, AFRP and steel reinforcement (Khalifa, 1999).
2.4 FRP Application on Structural Member

As the characteristics of FRP give promising results, FRP applications have been widely used in civil engineering arena. There are two types of FRP application where it involves FRP reinforcement used for new concrete structural members and FRP strengthening for existing structural members (Bank, 2006). Bank (2006) mentioned that the used of FRP for new structural members can be divided into three including FRP bars or grid reinforced concrete members, FRP tendons for prestressed concrete members and stay-in-place FRP formwork for reinforced concrete members. For FRP strengthening for existing structural members, it was divided into two major types namely strengthening and repair. FRP strengthening means the increase of original design structure’s capacity and strength while repair is where FRP was used to retrofit an existing and deteriorated structure and bring its strength and capacity back to the original as designed where it in fact a type of strengthening.

FRP bars are used as an alternative material to steel reinforcement that is subjected to potential corrosion condition. FRP bars are commonly used in severe
environmental condition such as coastal environments and water treatment plants. It includes dry-docks, sea walls, wharfs, box culverts, reinforced piles, floating piers, tank, facades and retaining walls (Gravina & Smith, 2008). Besides the application for corrosion purposes, FRP bars can be used as the main reinforcement for reinforced concrete members where magnetic transparency is required (El-Sayed & Soudki, 2010). Study and research on FRP bars replacing steel reinforcement has considered many parameters that affects the performance of the structure such as bond characteristic between FRP bars and concrete, ductility of FRP and moment distribution of members with FRP bars.

FRP tendons are used as an alternative material to steel tendons. FRP tendon is a material that has high tensile strength in the direction of the fibres and weak in the transverse direction (Elrefai, West & Soudki, 2006). Zou (2003) mentioned, when using FRP tendon in concrete members is the perceived reduction in ductility due to FRPs linear stress-strain relationship up to failure and its relatively low strain at rupture. The advantages of using FRP tendons include their non-corrosive and non-conductive properties, lightweight and high tensile strength. Therefore, in the last few decades, the application of FRP tendons was widely used in the construction of concrete bridges (Touakim & Karbhari, 2006).

The use of stay-in-place FRP formwork is a system that acts to reinforce the concrete after it has cured (Bank, 2006). FRP stay-in-place (SIP) form has been considered as an alternative to steel SIP form which it can reduce the construction cost and time. This is due to the reduced of transportation requirements and ease of construction where it is light in weight (Hanus, Bank & Oliva, 2009). FRP tubular SIP forms have also been used to manufacture beam and column members where they are also referred to as concrete-filled FRP tubes (Bank, 2006).

For strengthening reinforced concrete structure using FRP material, it is a technique to increase the load and displacement capacity of a member. This is to make the structure compatible with current existing building codes and regulations or due to the changes in the use of the structure itself (Bank, 2006). There are few types of strengthening using FRP which includes strengthening using externally bonded FRP sheets and plates. The research on strengthening existing structure has been widely
explore by many researchers since the application of FRP to replace steel plate as a strengthening material has been found to give better results in terms of corrosion resistance.

Repair of existing reinforced concrete structures using FRP is intended to increase the strength and ductility of damaged structures due to a few reason such as corrosion, design failure or seismic impact. This type of rehabilitation is an alternative solution besides other solution such as demolishing the structure. This is because some building such as historical building needs to be maintained because of its special characteristic and historical reason. Its advantages such as lightweight, high strength and corrosion resistance are some of the reason that makes them a suitable material for repair.

2.5 Shear Failure

The study of shear failure has received great attention among researchers on every aspect such as the factors that contribute to the shear failure, design aspect and others. Generally, shear failure is a diagonal tension failure that is brittle in nature and should be avoided (Wang, Salmon & Pincheira, 2007). In this few decades, using FRP is one of the solutions to overcome shear failure problem and since that extensive research using FRP in strengthening structural members has been conducted.

Due to the difference in mechanical properties between FRP and steel reinforcement, particularly the modulus elasticity, the shear strength of concrete members reinforced longitudinally with FRP bars may differ from that of members reinforced with steel. Although there is a guideline by ACI Committee 440 about the shear design for structures reinforced with FRP, many researchers has done further research about the shear design provisions. The ACI Committee 440 recommended that the shear design for reinforced concrete beams reinforced with FRP are based on the design formulas of members reinforced with conventional steel. ACI is currently in the process of revising its approach to the calculation of the concrete contribution to shear resistance of FRP reinforced concrete members.
Current design guidelines generally follow the format of conventional reinforced concrete design methods (Razaqpur et al., 2006). The test result of the investigation revealed that concrete shear strength of concrete beams reinforced with FRP bars to that of beams reinforced with steel is proportional to the cube root of axial stiffness ratio between FRP and steel reinforcing bars. The higher the reinforcement ratio or modulus of elasticity of the reinforcing bars, the higher the shear strength obtained.

2.5.1 Theory of Shear

Shear failure of reinforced concrete occurs due to a combination of stresses resulting from the applied shear force, bending moments and sometimes axial load as well as torsion. The behaviour of reinforced concrete in shear is very complex as the current code and design procedures are based on analysis of experimental results and model assumption rather than on the exact acceptable theory. The complexity are due to the non-homogeneity of material, nonlinearity of material, cracks, presence of reinforcement, load effects and many other things (Pillai, Kirk & Erki, 1999).

Shear cracks develop due to principal tensile stresses exceeding the tensile strength of the concrete (MacGregor & Bartlet, 2000). For the simple beam as shown in Figure 2.2, the bending moment M at section A-A causes compressive stresses in the concrete above the neutral axis (NA) and tensile stresses in the reinforcement as well as concrete below the neutral axis. An element located at the neutral axis generates a state of pure shear as shown in Figure 2.3 where there are no tensile or compressive stresses on the faces of the element and acting on 45° plane. This diagonal tension causes diagonal cracking, thus the failures in beams commonly referred to as ‘shear failures’ are actually tension failures at the inclined cracks (Wang et al., 2007).
Figure 2.2: Shear force and bending moment in a simply supported beam (Wang, et al., 2007)

Figure 2.3: A state of pure shear (Wang, et al., 2007)

For a homogeneous rectangular concrete beam, the flexural stress, $f_x$ and shear stress, $\nu$ at a point in the section distant $y$ from the neutral axis are given by:

$$f_x = \frac{My}{I}$$  \hspace{1cm} \text{(Equation 2.1)}

$$\nu = \frac{VQ}{Ib}$$ \hspace{1cm} \text{(Equation 2.2)}

Where:

$Q$ = The first moment of the area of section above the depth $y$, about the neutral axis

$I$ = Moment of inertia of the section

Figure 2.4 to 2.7 shows the flexural and shear stresses, principal stresses, stress trajectories and potential crack pattern of a simply supported beam respectively (Pillai et al., 1999). Concrete cracks when the principle tensile stresses exceed the tensile strength of the concrete. Vertical cracks occurred first due to flexural stresses and it starts at the bottom of the beam where the flexural stresses are the largest. Next, inclined cracks occurred at the end or near the support of the beam due to combined shear and flexure. These cracks must exist before a beam fails in shear. Some inclined cracks extended along main reinforcement towards the support and weakening the anchorage of the reinforcement (MacGregor & Bartlet, 2000).
Figure 2.4: Flexural and shear stresses (Pillai et al., 1999)

Figure 2.5: State of pure shear and principal stresses (Pillai et al., 1999)

Figure 2.6: Stress trajectories (Pillai et al., 1999)

Figure 2.7: Potential crack pattern (Pillai et al., 1999)
2.5.2 Behaviour of Beam without Stirrups

Figure 2.8 shows the shear mechanism acting in a beam without stirrups. A change in moment (thus shear transfer along the shear-span), can be by one of two mechanism (Stratford & Burgoyne, 2003):

i) Variation in the magnitude of the internal actions

ii) Variation in the lever-arm between the actions

Beam action describes shear transfer by changes in the magnitude of the compression-zone concrete and flexural reinforcement actions, with a constant lever-arm, requiring load-transfer between the two forces (ACI Committee 440, 2006). The transfer of shear in reinforced concrete without shear reinforcement occurs by a combination of the following mechanisms (Wang et al. 2007):

i) Shear resistance of the uncracked concrete, \( V_c \)

ii) Aggregate interlock force, \( V_a \) tangentially along a crack

iii) Dowel action, \( V_d \) (the resistance of the longitudinal reinforcement to a transverse force

iv) Arch action (on deep beams)

![Shear mechanism diagram](image)

Figure 2.8: Shear mechanism acting in a beam without stirrups (Wang et al., 2007)

The equilibrium of vertical forces is:

\[ V = V_c + V_a + V_d \]  

(Equation 2.3)

The proportion of shear transferred by the various mechanisms are in the range of 20 to 40% by the uncracked concrete, 33 to 50% by the aggregate interlock and 15 to 25% by
the dowel action (Sinha, 2002). The equation to calculate shear strength for beams without web reinforcement which is presented in ACI 318-08 (2008):

\[ V_a = V_c \]  

\[ V_c = \left( 1.9 \sqrt{f_c^t} + 2500 \frac{f_y V_{yd}}{f_y} \right) b_w d \leq 3.5 \sqrt{f_c^t} b_w d \]  

(Equation 2.4)  

(Equation 2.5)

in SI unit with \( f_c \), in MPa, ACI 318-08 gives:

\[ V_c = \left( 0.16 \sqrt{f_c^t} + 17 \frac{f_y V_{yd}}{f_y} \right) b_w d \leq 0.29 \sqrt{f_c^t} b_w d \]  

(Equation 2.6)

The equation is then simplified where the second term will be equal to about 0.008 \( \sqrt{f_c^t} \):

\[ V_c = 0.167 \sqrt{f_c^t} b_w d \]  

(Equation 2.7)

2.5.3 Behaviour of Beam with Stirrups

The strength of beams drops below the flexural capacity due to inclined cracking. Therefore, web reinforcement is used to ensure that the full flexural capacity can be achieved (MacGregor & Bartlett, 2000). Shear reinforcement of beams is usually provided by vertical stirrups or bent-up bar or combination of them. Shear reinforcement contributes to the strength of beam in the following ways (Sinha, 2002):

i) Carries a part of shear due to truss action

ii) Increases shear transfer by aggregate interlock by limiting the opening of the diagonal tension crack

iii) Increases the dowel action by providing support to the longitudinal bar which is being crossed by the shear crack

iv) Increases strength of concrete in region affected by arch action due to confinement of concrete by the closely spaced web reinforcement

Figure 2.17 below shows the redistribution of internal shear in beam with shear reinforcement.
Prior to incline cracking, the strain in stirrups is equal to the corresponding strain of the concrete and since concrete cracks at a very small strain, the stress is the stirrups prior to inclined cracking will not exceed 20 to 40 MPa. Therefore, stirrups do not prevent inclined cracks from forming but they come into play after the cracks have formed (MacGregor & Bartlett, 2000). For beam with stirrups, the shear strength which is the nominal strength is calculated as follows:

\[ V_c = V_c + V_s \]  \hspace{1cm} (Equation 2.8)

\[ V_s = A_s f_y \sin \alpha \]  \hspace{1cm} (Equation 2.9)

For beam where the stirrups are perpendicular to its longitudinal axis, which is \( \alpha = 90^\circ \):

\[ V_s = \frac{A_s f_y d}{s} \]  \hspace{1cm} (Equation 2.10)
Factors Affecting the Contribution of the FRP Sheets to Shear Capacity

There are a few factors that affect the contribution of the FRP sheets to shear capacity, including wrapping schemes, fiber orientation, end anchor, biaxial reinforcement and bonding of FRP strips.

2.11 Wrapping Schemes

There are three types of wrapping schemes of FRP used to increase the shear capacity of rectangular beam or column as shown in Figure 2.18 (ACI Committee, 2008). Figure 2.18(a) shows a fully wrapping scheme which covers all four sides of members. This type of wrapping is the most efficient and commonly used to strengthen column while the application on beams is rather impractical because of the presence of monolithic slab. Therefore, wrapping three sides around the member as illustrated in Figure 2.18(b) is a solution to overcome the weakness. Another type of wrapping scheme of the FRP is two sides bonding as shown in Figure 2.18(c) where this method is the least efficient among the three types because of the weakness in term of anchorage of the FRP sheets (ACI Committee, 2008).

Figure 2.10: Types of wrapping schemes for shear strengthening using FRP laminates (ACI Committee, 2008)
2.6.2 Fiber Orientation

Besides wrapping schemes of the CFRP strips, the CFRP strips orientation is also one of the factors that affect the effectiveness of the shear contribution of the CFRP. A study by Norris et al (1997) concluded that when the CFRP fibers were placed perpendicular to cracks of the tested specimens, a significant increase in stiffness and strength was observed and a brittle failure occurred due to concrete rupture as a result of stress concentration near the ends of the CFRP. In contrast, when the CFRP fibers were placed obliquely to the cracks of the tested specimens, a smaller increase in strength and stiffness was observed. However, the modes of failure was more ductile and preceded by warning signs such as snapping sounds or peeling of the CFRP. Apart from that, a study by Li et al. (2006) has found out that the strength, ductility and modes of failure of FRP wrapped to a cylinder depends on the fiber orientation and wall thickness.

Another research by Taljsten (2003) involving a series of strengthening beams for shear with different fiber orientation as shown in Figure 2.19 to Figure 2.21. The beams were strengthened with CFRP fabrics with different angle and weight of the fabric which in direct proportion to the thickness of the sheet. From his study, Taljsten concluded that concrete beams can be strengthened for shear and the fabrics laminates should be placed perpendicular to the shear crack if possible. The study also mentioned that the thinner the fibre used, the better utilisation of the fabric. The most common orientations used to prevent shear failure are 0 degree, 90 degree and 45 degree. For bi-directional fiber direction, the fiber orientations are 0/90 degree and 45/135 degree orientation.

![Figure 2.11: Fiber orientation, θ = 90°](image)

Figure 2.11: Fiber orientation, \( \theta = 90^\circ \)
2.6.3 End Anchor

In strengthening reinforced concrete beam by wrapping three sides of the beam, use end anchor, the FRP composite could fail in debonding from the concrete. The using of end anchor has been proved to increase the shear capacity of concrete beam externally strengthened with FRP (Khalifa, 1999). The using method has been done with steel plates and bolts as shown in Figure 2.22.
4.4 Biaxial reinforcement

Sheets are available in a variety of configurations including uni-directional, bi-directional, and random. Uni-directional FRP sheets have the fibers aligned in only one direction. On the other hand, bi-directional FRP sheet has the fibers aligned at right angles to each other. For this type of FRP sheets, it can be placed in horizontal and vertical directions where the fibers are in perpendicular directions and act as reinforcements as well as anchors (Balaguru, Nanni & Giancaspro, 2009). For fibers in random configuration, the fibers are randomly distributed and are in plane.

4.5 Spacing of FRP Strips

Spacing of FRP strips is defined as the distance between the centerline of the strips and should not exceed the sum of d/4 plus the width of the strip (ACI Committee, 2008). Nanni & Nanni (2000) mentioned that the spacing of CFRP strips should not be so close in order to allow the full formation of a diagonal crack without intercepting a strip. Figure 2.25 shows the width, \( w_f \) and spacing, \( s_f \) of the strips. For continuous sheets, the spacing of strips are equal to width of strips (the ratio should be unity). The use of continuous sheets is however discouraged because the risk of it can prevent migration of moisture (ACI Committee, 2008).

![Figure 2.15(a) and 2.15(b): Vertical and inclined CFRP strips](image-url)
Theoretical Models on the Contribution of CFRP Sheets to Shear Capacity

Determination of the contribution of FRP reinforcement to shear capacity has been through the development of theoretical models which verified by experimental results as well as modeling using finite element analysis. In most of the theoretical models, the assumption made is the FRP reinforcement is treated the same way as the internal steel reinforcement in terms of its contribution to shear capacity. A model by Triantafillou in 1998 and Khalifa et al. in 1999 and 2002 are among the models that were referred by researchers before came out with their own modified design proposal.

Khalifa and Nanni Model (2000)

The proposed model was based on two design approach where modes of failure is the design criteria to be taken into account which is CFRP rupture and delamination of the FRP from the concrete surface. The first design approach was based on the effective stress of the FRP where it is only applicable if the failure is governed by CFRP rupture. On the other hand, the second design approach was based on the bond mechanism between the CFRP sheet and the concrete surface where the failure is governed by CFRP sheet delamination.

Equation 2.14 by Triantafillou was written in Eurocode format and therefore, Khalifa et al. has rewritten the equation in ACI format as follows:

\[
\frac{A_f f_b (\sin \beta + \cos \beta)d_f}{s_f} = \phi_f \\
(Equation 2.17)
\]

Where the modulus elasticity, \(E_f\) times the effective strain, \(\varepsilon_{ef}\), \((E_f \varepsilon_{ef})\) was replaced with the effective stress, \(f_b\). From the equation, the area of CFRP, \(A_f\) is equal to thickness of the FRP, \(t_f\), times the width of the FRP, \(w_f\). The thickness of the CFRP is multiply by 2 to represent the sheets on both sides. On the other hand, for continuous sheets, the width of the FRP should be the same value with the spacing of the CFRP strips, \(s_f\). The area of the CFRP may be expressed as follows:
\[\Delta f = 2\Delta \nu f \quad ; \quad \text{(Equation 2.18)}\]

The next step is to determine the effective stress of the CFRP, \(f_e\). In order to use the factors of various types of CFRP sheets, a reduction factor, \(R\) is applied to the effective stress of the CFRP should be smaller than its ultimate stress, \(f_{tu}\) as follows:

\[f_e = Rf_{tu} \quad ; \quad \text{(Equation 2.19)}\]

\(R\) is the ratio of effective strain or stress to its ultimate strain or stress and a reduction of Equation 2.15 and Equation 2.16 by Khalifa et. al was made based on an assumption that \(\rho E_f\) does not exceed 1.1 GPa in all analyzed data from literature review and their own experimental results (Khalifa et. al, 1998). The reduction factor or constant depends on the modes of failure whether the failure is CFRP rupture or CFRP debonding. \(R\) is then calibrated in all available test data and therefore for failure which is CFRP rupture, the reduction factor was established as a function of \(\rho E_f\) and for 1.7 GPa, \(R\) is expressed as follows:

\[R = 0.56(\rho E_f)^2 - 1.22(\rho E_f) + 0.78 \quad \text{(Equation 2.20)}\]

On the other hand, for failure which based on CFRP debonding, the shear capacity of the CFRP is a function of CFRP axial rigidity, concrete strength, effective width of the CFRP sheet and bonded surface configurations. FRP debonding is an important consideration especially when beam is not fully wrapped (four sides bonding) with CFRP sheet. For failure governed by CFRP debonding, the reduction factor is given as follows:

\[R = \frac{(f_c/E_f)^{2/3} w_{fe}}{\varepsilon_{tu} d_f} \left[738.93 - 4.06(\rho E_f)\right] \times 10^{-6} \quad \text{(Equation 2.21)}\]

This equation is applies for CFRP with the axial rigidity, \(\rho E_f\) ranging from 20 to 100 GPa (kN/mm). To gain the effective depth, \(w_{fe}\), the wrapping scheme has to be considered with the crack angle of 45°.
the effective depth is expressed as follows:

\[ d_e = \frac{d}{2} \times L_e \] \hspace{1cm} (Equation 2.22)

\[ d_e = \frac{d}{2} \times 2L_e \] \hspace{1cm} (Equation 2.23)

Apart from that, there is an upper limit of determining \( R \) in order to maintain the integrity of the concrete because beyond the limit, the shear failure is more on the 
interlock of the concrete. From previous researches, the upper limit of \( R \) is 
\[ 0.003 \times d \] where the ultimate strain in CFRP materials cannot be greater than 
\( 0.003 \text{ mm/mm} \). From these three conditions, the lowest value is taken as the reduction 
for shear contribution of CFRP reinforcement.

Another factor which has to be considered in determining the shear contribution of 
CFRP reinforcement is spacing of the CFRP strips. Therefore, if a CFRP strip is 
\( s \) spacing of the strips should be determined using follows equation:

\[ s \leq \frac{w}{4 + \omega d/4} \] \hspace{1cm} (Equation 2.24)

In order to follow the requirement of ACI 318 on the total shear strength, a 
modification has been made as suggested below:

\[ 2(\sqrt{f} \cdot c) b_w d \]

\[ \frac{\psi s}{\lambda} \leq V_s \] \hspace{1cm} (Equation 2.25)

The Equation 2.14 may be rewritten in Eurocode format as follows:

\[ V_s = \frac{A_e \left( \frac{f_y}{2} \right) (0.9d)(1 + \cot \beta) \sin \beta}{s_f} \leq \left[ V_{rd2} - (V_{rd1} + V_{wd}) \right] \] \hspace{1cm} (Equation 2.26)

The partial safety factor, \( \gamma_f \), was suggested to be taken as equal to 1.3.
REFERENCES


ACI Committee 318, Building code requirements for structural concrete (ACI 318-08), American Concrete Institute, Detroit, 2008.


