SHEAR STRENGTHENING OF PRE-CRACKED AND NON PRE-CRACKED REINFORCED CONCRETE CONTINUOUS BEAMS USING BI-DIRECTIONAL CFRP STRIPS

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ABSTRACT

Shear failure of a reinforced concrete beam is catastrophic where it occurs suddenly and without any warning. The use of FRP sheet as a strengthening and repairing material is an effective method to enhance the shear capacity of the beam. Extensive researches have been conducted on the shear strengthening of reinforced concrete simply supported beams using FRP composites. However, strengthening continuous beams in shear have received very little attention among the researchers although most of the existing structures are in the form of continuous condition. Furthermore, there are restraints to add shear reinforcement to the existing reinforced concrete beams when beams are part of the floor-beam system. In the design guideline by ACI 440 Committee mentioned that the existing theoretical model have not been confirmed to be use for strengthening in negative moment region which existed in continuous beam. Therefore, in order to address the problem, a study on shear strengthening of reinforced concrete continuous beam using CFRP strips was conducted. An experimental work on 14 full-scale reinforced concrete continuous beams with a size of 150x350x5800mm was carried out. Simulation using finite element software ATENA v4 and theoretical analysis was also conducted. The variables involved a number of CFRP strips layers (one and two layers), wrapping schemes (four sides and three sides), orientation of CFRP strips (0/90 and 45/135 degree) and shear span to effective depth ratio, $\frac{a_s}{d}$ (2.5 and 3.5). The type of FRP used was bi-directional CFRP strips. Two beams were un-strengthened and treated as the control specimens whilst the other 12 beam were wrapped with CFRP strips. From the experimental results, all beams failed in shear as expected. Beams wrapped with CFRP strips recorded shear capacity enhancement of around 10.12% to 53.74% compared to the control specimens. Beam wrapped with two layers of CFRP strips at four sides of the beam recorded the highest shear enhancement. Simulation study also showed similar behaviour in terms of shear capacity and crack patterns. Three existing theoretical models; ACI 440, Khalifa and Nanni and fib models were adopted for theoretical comparison of shear capacity contributed by CFRP, $V_f$ while for shear capacity contributed by concrete, $V_c$ and stirrups, $V_s$, the equation from ACI 318-08, BS8110 and EC2 was adopted. The ACI 440 model had shown the closer value with the experimental results and a modified ACI 440 model was proposed on the effective strain limit and bond-reduction coefficient.
ABSTRAK

Kegagalan ricih bagi rasuk konkrit bertetulang adalah merbahaya dimana ianya berlaku secara tiba-tiba tanpa sebarang amaran. Penggunaan FRP sebagai bahan pengukuhan dan pemulihan adalah satu kaedah yang telah diketahui efektif bagi meningkatkan kekuatan ricih rasuk tersebut. Kajian yang mendalam telah pun dilakukan ke atas pengukuhan ricih rasuk konkrit bertetulang disokong mudah. Bagaimanapun, kajian ke atas pengukuhan ricih bagi rasuk selanjar masih tidak mendapat perhatian yang meluas sedangkan kebanyakan struktur sedia ada adalah dalam bentuk selanjar. Terdapat juga halangan bagi menambah tetulang ricih pada rasuk konkrit bertetulang yang sedia ada apabila rasuk adalah sebahagian daripada sistem papak-rasuk. Di dalam garis panduan rekabentuk yang dikeluarkan oleh ACI 440 Committee memberitahu bahawa model analitikal sedia ada masih belum dipastikan bagi digunakan untuk pengukuhan rasuk dibahagian momen negatif yang mana ianya wujud pada rasuk selanjar. Oleh itu, satu kajian telahpun dijalankan terhadap pengukuhan ricih rasuk selanjar dengan menggunakan jalur-jalur CFRP. Satu kajian makmal terhadap 14 rasuk selanjar konkrit bertetulang berskala penuh dengan saiz 150x350x5800mm telahpun dijalankan beserta simulasi menggunakan perisian unsur terhingga ATENA v4 dan analisis teori. Antara pembolehubah yang terlibat ialah bilangan lapisan CFRP, skim balutan, orientasi jalur CFRP dan nisbah rentang ricih terhadap kedalaman berkesan. Dua rasuk tidak diperkukuhkan dan diambil sebagai rasuk kawalan manakala 12 rasuk yang selebihnya diberi balut dengan jalur-jalur CFRP. Daripada keputusan eksperimen, semua rasuk gagal dalam ricih seperti yang telah dijangkakan. Rasuk yang diperkukuhkan dengan CFRP mencatatkan peningkatan kekuatan ricih dalam lingkungan 10.12% - 53.74%. Rasuk yang dibalut dengan dua lapis jalur CFRP mencatatkan peningkatan ricih yang tertinggi. Kajian simulasi juga menunjukkan kelakuhan yang sama dari segi kekuatan ricih dan corak keretakan. Tiga model teori yang sedia ada ialah ACI 440, Khalifa & Nanni dan fib digunakan untuk perbandingan secara teori bagi kapasiti ricih oleh CFRP, $V_t$ manakala bagi kapasiti ricih oleh konkrit, $V_c$ dan tetulang ricih, $V_s$, tiga persamaan daripada ACI 318-08, BS8110 dan EC2 digunakan. Model ACI 440 menunjukkan nilai teori yang lebih hampir dengan ujikaji makmal dan satu pengubahsuaian terhadap model tersebut telapun dicadangkan ke atas had keterikan berkesan dan pembolehubah pengurangan-ikatan.
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<tr>
<td>°</td>
<td>Degree of angle</td>
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<tr>
<td>Ψ</td>
<td>FRP strength reduction factor</td>
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<tr>
<td>φ</td>
<td>Strength reduction factor</td>
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<tr>
<td>γ_t</td>
<td>Partial safety factor</td>
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<tr>
<td>α</td>
<td>Angle of orientation of shear reinforcement</td>
</tr>
<tr>
<td>θ</td>
<td>Angle of orientation of FRP reinforcement</td>
</tr>
<tr>
<td>β</td>
<td>Angle of orientation of FRP reinforcement</td>
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<tr>
<td>ε</td>
<td>Strain</td>
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<td>ε_{fu}</td>
<td>The ultimate strain in the FRP</td>
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<td>ε_{fc}</td>
<td>The effective strain in the FRP</td>
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<tr>
<td>Δ</td>
<td>Deflection</td>
</tr>
<tr>
<td>µ</td>
<td>Micro</td>
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<tr>
<td>ρ_w</td>
<td>Steel reinforcement ratio</td>
</tr>
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<td>ρ_f</td>
<td>FRP reinforcement ratio</td>
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<tr>
<td>a/d</td>
<td>Shear span to effective depth ratio</td>
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<tr>
<td>A_s</td>
<td>Area of longitudinal tension reinforcement</td>
</tr>
<tr>
<td>A_f</td>
<td>Area of shear reinforcement</td>
</tr>
<tr>
<td>A_{fr}</td>
<td>The area of FRP shear reinforcement</td>
</tr>
<tr>
<td>A_{fpe}</td>
<td>The area of FRP shear reinforcement</td>
</tr>
<tr>
<td>b</td>
<td>Width of section</td>
</tr>
<tr>
<td>b_w</td>
<td>Web width</td>
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<tr>
<td>d</td>
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<tr>
<td>d_f</td>
<td>The effective depth of FRP at section</td>
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<tr>
<td>E_f</td>
<td>The modulus of elasticity of the FRP</td>
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<tr>
<td>I</td>
<td>Moment of inertia</td>
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<tr>
<td>f_s</td>
<td>Flexural stress</td>
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<tr>
<td>f_c</td>
<td>Concrete compressive stress</td>
</tr>
<tr>
<td>f_y</td>
<td>Yield strength of longitudinal tension reinforcement</td>
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<td>f_v</td>
<td>Yield strength of shear reinforcement</td>
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<tr>
<td>f_{fe}</td>
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The ultimate stress in the FRP

The bond-reduction coefficient

The modified bond-reduction coefficient

Modification factor for concrete strength

Modification factor for wrapping scheme

Kilo-Newton

The effective length of FRP reinforcement

Number of layer of FRP reinforcement

Bending moment

Factored moment at section

Ultimate load of simulation results

Ultimate load of experimental results

Spacing centre-to-centre between reinforcement

Spacing centre-to-centre between FRP strips reinforcement

Thickness of FRP sheet

Shear stress

Shear force

Shear resistance of the uncracked concrete

Aggregate interlock force

Dowel action

Nominal shear strength

Shear strength contributed by concrete

Shear strength contributed by shear reinforcement

Shear strength contributed by CFRP

Shear resistance of uncracked concrete

Interface shear transfer

Ultimate shear strength of web steel

The shear resistance of a member with shear reinforcement

The FRP contribution to shear capacity

Factored shear force at section

Shear strength of simulation results

Shear strength of experimental results

Shear strength of theoretical value

Ultimate shear strength
\( V_{f,\text{exp}} \) Shear strength of experimental results contributed by CFRP
\( V_{f,\text{theory}} \) Shear strength of theoretical value contributed by CFRP
\( w_f \) Width of FRP strips
\( w_{fe} \) The effective width of FRP strips
C1, C2, C3, C4 Strain gauge at concrete surface
F1, F2, F3, F4 Strain gauge at CFRP strips
S1, S2, S3, S4 Strain gauge at stirrups
# LIST OF ABBREVIATIONS

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<td>2D</td>
<td>Two dimensional</td>
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<tr>
<td>3D</td>
<td>Three dimensional</td>
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<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>AFRP</td>
<td>Aramid Fibre Reinforced Polymer</td>
</tr>
<tr>
<td>ATENA</td>
<td>Advanced Tool for Engineering Nonlinear Analysis</td>
</tr>
<tr>
<td>CEB</td>
<td>The Euro-International Committee for Concrete</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon Fibre Reinforced Polymer</td>
</tr>
<tr>
<td>fib</td>
<td>The International Federation for Structural Concrete</td>
</tr>
<tr>
<td>FIP</td>
<td>The International Federation for Prestressing</td>
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<tr>
<td>FEM</td>
<td>Finite Element Method</td>
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<tr>
<td>FRP</td>
<td>Fibre Reinforced Polymer</td>
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<td>GFRP</td>
<td>Glass Fibre Reinforced Polymer</td>
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<tr>
<td>HFRP</td>
<td>Hybrid Fibre Reinforced Polymer</td>
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<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transducer</td>
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<tr>
<td>N.A.</td>
<td>Neutral Axis</td>
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<td>TRC</td>
<td>Textile Reinforced Concrete</td>
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<td>UTM</td>
<td>Universal Testing Machine</td>
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<td>SBETA</td>
<td>StahlBETonAnalyse (the analysis of reinforced concrete in German language)</td>
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CHAPTER 1

INTRODUCTION

1.1 Introduction

In this few decades, the use of Fibre Reinforced Polymer (FRP) laminates as a strengthening material has become a well known method in civil engineering. However, prior to civil engineering, in the early days back to 1930s, FRP laminates had been widely used in many areas such as aerospace, transportation, maritime and electrical (ACI 440, 2006). In civil engineering, old buildings such as historical building that need to be preserved are among the reasons why FRP has been widely used. FRP laminates are chosen because of their good 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 FRP composites materials are an alternative solution to strengthen existing structures (Chajes et al., 1995; Norris et al., 1997; Grace et al., 1998; Khalifa & Nanni 2000; Taljsten 2003; Adhikary & Mutsuyoshi 2004;). Old buildings and existing structures have motivated many researchers and organizations to find alternative materials and techniques to restore the deteriorating and deficient structures (Taljsten, 2003). In Malaysia, FRP laminates were used to strengthen and repair the structure of the Middle Ring Road 2 (MRR2) in Kepong when the flyover was seriously damaged and cracks were clearly seen at the piers and girders.

Past researches had shown great interest in shear and flexural strengthening on reinforced concrete structures. Unlike flexural behaviour of cracked reinforced concrete beams which can be well predicted, the prediction of shear behaviour of reinforced concrete beams is a tough task due to its complexity on shear transfer mechanism (El-Ariss, 2007). Bank (2006) pointed out that the preferable modes of
failure in a reinforced concrete beam is yielding of the tension reinforcement and followed by crushing of the concrete in the compression zone.

Shear resistance of a reinforced concrete beams is a contribution from the shear transfer in the compression zone, aggregate interlock across the crack face, stirrups crossing the shear crack and the dowel action of longitudinal reinforcing bars crossing the crack in the concrete. The shear failure of reinforced concrete beam could be by diagonal tension failure or shear compression failure (Balaguru et al., 2009). In order to strengthen or repair structure with shear defect, composites materials have been widely used. For shear strengthening using composite materials, the composites can be in many forms which include sheets, plates or bars along the depth of the beam or perpendicular to the potential shear cracks. Shear strengthening has also been found to improve the ductility because of the partial confining provided by the strengthening systems (Balaguru et al., 2009).

1.2 Problem statement

Structures that fail in shear are more dangerous than flexural failure because shear failure occurs suddenly and without any warning (Khalifa & Nanni, 2000, Zhang and Hsu, 2005; Jayaprakash et al., 2008). Shear failure is a diagonal tension failure that is brittle in nature and should be avoided (Wang et al., 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 complexion is due to the non-homogeneity of material, nonlinearity of material, cracks, presence of reinforcement, load effects and the environment (Pillai et al., 1999).

Therefore, it is important to strengthen a structure that is deficient in shear. Khalifa & Nanni (2000) observed that many existing reinforced concrete structures are deficient in shear and are of need to be repaired. Reasons for these deficiencies are insufficient shear reinforcement, increase of service load and corrosion of the reinforcement. Historically, the strengthening and repairs of existing concrete structures have been done by using conventional techniques such as external bonded steel plates, steel or concrete jackets and external post-tensioning (ACI Committee 440, 2002). However, these conventional techniques require maintenance where a
steel plate has the risk of corrosion due to the environment. As this problem arises, a lot of studies have been carried out to find a solution to overcome the problem.

Generally, most of investigations carried out experimentally by previous researchers focused on shear strengthening of reinforced concrete beams that are simply supported. In reality, 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 beams are in the form of continuous condition. Furthermore, there are restraints to add shear reinforcement to existing reinforced concrete beams when beams are a part of floor-beam system. For that reasons, FRP has been seen as the solution to overcome the problems. Therefore, an application of composites material has been emphasized to extend the service life of existing concrete structures. In this research, in order to fill the gap, 14 continuous beams strengthened and repaired using CFRP strips have been casted and tested where the results were then analyzed and compared with simulation and theoretical study.

1.3 Objective

The main aim of this study was to investigate the effectiveness of using externally bonded bi-directional Carbon Fibre Reinforced Polymer (CFRP) composites as a shear strengthening and repair technique for reinforced concrete continuous beams. At the end of the research work, a theoretical model for the shear capacity of these beams is expected to be established. To ensure the success of this research work, the following objectives were outlined:

a) To investigate the shear behaviour of externally bonded reinforced concrete continuous beams using CFRP strips with different parameters such as shear span to effective depth ratio, number of layer, wrapping scheme and orientation of CFRP strips.

b) To compare and validate a finite element model of reinforced concrete continuous beams strengthened and repaired using CFRP strips with the experimental results.
c) To propose a modified empirical equation for the shear strength contribution of CFRP ($V_f$) for reinforced concrete continuous beams externally bonded with CFRP strips.

1.4 Scope of study

This study focused on shear strengthening and repair of reinforced concrete continuous beam using externally bonded CFRP sheet. Therefore, in order to achieve the above-mentioned objectives, the following scopes and limits were outlined:

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

ii) All specimens were designed to fail in shear. Sufficient flexural reinforcement was employed on the specimens to avoid any failure due to flexure.

iii) A total of 14 specimens were casted and tested including two control specimens and 12 reinforced concrete beams strengthened and repaired with different schemes of CFRP strips.

iv) The specimens were divided into two groups of different shear span to effective depth ratio ($a_v/d$) i.e. 2.5 and 3.5 respectively.

v) All specimens had identical design detail with similar concrete compressive strength of 30N/mm², main reinforcement tensile strength of 460N/mm² and stirrups tensile strength of 250N/mm².

vi) The bi-directional CFRP sheet was selected for the experimental programme.

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

viii) The parameter involves shear span to effective depth ratio, number of layer, wrapping scheme and orientation of CFRP strips.

ix) The experimental performance observation consisted of the deflection of the specimen, ultimate load, loading history, strain of stirrup, concrete surface and CFRP strips as well as modes of failure and crack pattern of the specimens.
x) The simulation using of finite element analysis software (ATENA V4) was conducted to analyze the theoretical behaviour of the specimens, to compare and the results were then verified using the experimental results obtained.

Figure 1.1 shows the flow chart of the research work conducted. Initially, a literature review was carried out focusing on the background of the CFRP composites, shear strengthening of reinforced concrete continuous beams and the theoretical modelling from previous researchers. This was followed by a series of laboratory work encompassing the fabrication and instrumentation of the continuous beams. This step also involved the installation of CFRP strips as well as testing procedure of the continuous beams. Upon completion of the laboratory work, an analysis of the data collected from the testing procedure was conducted. The data were then compared with the theoretical investigation from the computational study using finite element software (ATENA V4). Finally, an empirical equation for the shear capacity of strengthened and repaired reinforced concrete continuous beams using externally bonded CFRP composites were established.
Proposing a modified empirical equation for shear capacity of continuous beam strengthened using CFRP strips

Figure 1.1: Flow of research
1.5 Research significance

FRP composites as strengthening and repairing materials have been acknowledged as an effective method to increase the load capacity of reinforced concrete beams. This was proved from extensive studies done by many researchers (Chajes et al., 1995; Hollaway & Leeming, 1999; Khalifa & Nanni, 2002; Zou, 2003; Taljsten & Blanksvard, 2007; Jayaprakash et al., 2008). Most of the studies focused on strengthening of un-cracked reinforced concrete beam and limited studies were done on repair of defected or cracked reinforced concrete beam. However, Jayaprakash et al., (2008) had conducted an experimental work on strengthening un-cracked reinforced concrete simply supported beam and repair of pre-cracked reinforced concrete simply supported beam. Their findings revealed that the pre-cracked beam could perform as good as the initially strengthened beam due to the presence of CFRP sheet.

The current design codes on strengthening using FRP composites were developed based on experimental results of un-cracked reinforced concrete beam (fib, 2001; ACI Committee 440, 2008). Therefore, more experimental data on repair of cracked reinforced concrete beam are needed to verify the design codes to be applied on cracked reinforced concrete beam. As mentioned in the problem statement of this study, shear strengthening in negative moment region has not been confirmed by the current design code by ACI Committee 440 (2008). Therefore, in order to fill the gap on strengthening and repairing of cracked reinforced concrete beam and shear strengthening in negative moment region, this study which was focusing on shear strengthening and repair of un-cracked and pre-cracked reinforced concrete continuous beam was conducted. An experimental work followed by simulation study using finite element software (ATENA V4) and theoretical analysis was executed where a modification on the current design code by ACI Committee 440 was proposed.

1.6 Structure of thesis

Chapter 1 presents an introduction to the thesis including the problem statement, objectives, scope of research and the significance of this research.
Chapter 2 reviews previous research work on shear strengthening and repair of reinforced concrete beams externally bonded with Carbon Fibre Reinforced Polymer (CFRP) strips. In-depth study on the existing theoretical models for the prediction of the shear capacity of the strengthened beams was also reviewed.

Chapter 3 presents the research methodology of experimental work on 14 reinforced concrete continuous beams strengthened and repair using CFRP strips. This chapter also presents the material properties, specimen preparation and test set-up and instrumentation. This chapter also presents the methodology of finite element modeling using ATENA v4 software.

Chapter 4 presents the experimental results of shear strengthening of reinforced concrete continuous beams using CFRP strips. The data analysis includes the discussion on ultimate load, crack pattern, shear force – deflection profile and shear force – strain behaviour. The strain was observed on stirrups, concrete surface and CFRP strips.

Chapter 5 presents the simulation results and its validation of the experimental data. In addition, this chapter also shows the theoretical values using three existing model; ACI 440, Khalifa & Nanni and fib model. Comparison between experimental results and theoretical values was also discussed. Finally, a modified empirical equation on shear strength contribution of CFRP for continuous beams was proposed.

Chapter 6 summarizes the experimental, simulation and theoretical result. This chapter concludes the major findings of the research and proposed recommendation for future research.
CHAPTER 2

LITERATURE REVIEW

2.1 Historical background of Fibre Reinforced Polymer (FRP)

The use of basic materials in the fabrication of dwellings including mud, straw, wood and clay had been widely used in the early ages. While the concept of composites has been in existence for several millennia, the incorporation of Fibre Reinforced Polymer (FRP) composites technology into the industrial world is less than a century old. The true age of plastics emerged just after 1900, with chemists and industrialists taking bold steps to have plastics (vinyl, polystyrene, and plexiglass) mimic and outdoor natural materials. The first known FRP product was a boat hull manufactured in the mid-1930s as part of a manufacturing experiment using a fibreglass fabric and polyester resin laid in a foam mold (ACI Committee 440, 2006). From this first invention, FRP has been widely used in many areas and industries such as aerospace, marine, transportation and electrical due to its extraordinary strength and stiffness properties.

One of the advantages of using FRP is its corrosion-resistance characteristic which contributes to its tremendous used in the US Air Force and Navy arena. In 1960s, the British and U.S. naval forces developed minesweeper ships using FRP composites, as these materials are superior in aggressive marine environment and are non-magnetic in nature (ACI Committee 440, 2006). Since that, FRP has received an extensive attention among the researchers to develop new products by using FRP as part of the materials. In recent years, the benefit of FRP composites especially corrosion resistance characteristics has contributed to its application in the public sector.
While the majority of the historical and durability data of FRP composite installations comes from the aerospace, marine and corrosion resistance industries; FRP composites have been used as a construction material for several decades. FRP composite products were first demonstrated to reinforce concrete structures in the mid-1950s (ACI Committee 440, 2006). Due to its excellent performance, the use of FRP was expanded to restore historic buildings and other structural applications.

2.2 Advantages of FRP composites

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 defense area, FRP composites also provide many advantages in civil engineering. With many serious problems such as the service life of the structures, corrosion of reinforcement, design faults and improper planning, FRP is believed to be one of the solutions.

The bonding of FRP plate on the tensile face has been proven to be an effective method to increase both the strength and stiffness of concrete members (Leung & Pan, 2005). Depending on the products and applications, FRP materials 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. The use of FRP also reduces the maintenance costs because of resistance to deicing salts and other corrosive agents (ACI Committee 440, 2006). The other advantages of FRP composites are; they can reduce field installation time by using engineered system packaging, reduce traffic delays because of the faster construction, increase reliability by pre-engineered systems, enhance the durability and fatigue characteristics and increase the service life of the structure. Besides that, because on the innovative and efficient installations, the engineering value of the products and systems are improved (ACI Committee 440, 2006).
2.3 Disadvantages of FRP composites

Using FRP composites has its disadvantages. Taljsten & Blanksvard (2007) highlighted that since the application of FRP in building industries has been around only for 10 years, there were insufficient data to verify that FRP has good long-term properties. The working environment while handling FRP composites is also important as any mishandling of these materials may cause injuries to workers. FRP composites have low ductility where the stress-strain relationship is linear that could cause a sudden and brittle failure (Kodur & Baingo, 1998).

2.4 Fibre Reinforced Polymer constituents

Fibre Reinforced Polymer is a composite material, tailored by a large number of thin high strength fibres embedded in a plastic resin (ACI Committee 440, 2006). FRP is defined as a polymer matrix, either thermoset or thermoplastic, that is reinforced with a fibre or other reinforcing material with a sufficient aspect ratio (length-to thickness) to provide a discernible reinforcing function in one or more directions (ACI Committee 440, 2006). Cusson & Xi (2002) have stated that a fibre-reinforced polymer or FRP is an advanced composite or material system. It is defined as a solid material which is composed by 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 an excellent tensile strength in the direction of the fibres and negligible strength in the transverse direction. These fibres are named as uni-directional 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 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 of, 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 polymer 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.

2.4.1 Types of fibre reinforcement

In civil engineering, there are three types of fibres commonly used; 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) and width length nearly 100 times its diameter (Cusson & Xi, 2002). Glass fibres are produced by extruding molten mass through an orifice of 0.79-3.18mm in diameter followed by drawing through fine opening of 3-20µm in diameter (ACI Committee 440, 2006). Glass fibres are produced by extruding molten mass through an orifice of 0.79-3.18mm in diameter followed by drawing through fine opening of 3-20µm 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 compared to 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 a good electrical and thermal insulation materials. Apart from that, glass fibre is also used for radar antenna applications due to its characteristic 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 of above
1000°C (1830°F) (ACI Committee 440, 2006). 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 are some of the advantages of carbon fibres. 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 (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 are 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).

![Stress-strain relationship between CFRP, GFRP, AFRP and steel reinforcement](image)

**Figure 2.1:** Stress-strain relationship between CFRP, GFRP, AFRP and steel reinforcement (Khalifa, 1999)
2.4.2 Polymer matrix: resins

Resins can be classified into two groups: thermosetting and thermoplastic (ACI Committee 440, 2006). Thermoset resins are liquids or low melting point. They are cured with a catalyst, heat or a combination of the two. Once thermoset resins are cured, they cannot be converted back to their original liquid form. Cured thermoset will not melt and flow but it will soften when heated (ACI Committee 440, 2006).

For thermoplastic resins, they can be repeatedly softened at high temperature. Thermoplastic resins become soft when heated and may be shaped or molded while in a heated semi-fluid state and vice versa, they become rigid when cooled (ACI Committee 440, 2006). Thermoset resins are used for structural purposes for their ability to undergo a chemical reaction when cured (Cusson & Xi, 2002). The most common thermosetting resins used in composites are polyesters, epoxies, vinylesters and phenolics.

Polyesters are versatile because of their capacity to be modified or tailored during the building of the polymer chains. The principal advantage of these resins is a balance of properties (including mechanical, chemical and electrical) dimensional stability, cost and ease of handling or processing (ACI Committee 440, 2006). Epoxy resins are the most widely used and accepted as structural adhesive available commercially since 1940s. Epoxy resins are used with a number of fibrous reinforcing materials, including glass, carbon and aramid (ACI Committee 440, 2006). It has the ability to produce a continuous bond between fibre reinforced polymer (FRP) and concrete to ensure that full composite action is developed by the transfer of shear stress across the thickness of the adhesive layer.

Vinylesters are developed to combine the advantages of epoxy resins with those of unsaturated polyester resins (ACI Committee 440, 2006). The advantages of vinylesters are they offer mechanical toughness and excellent corrosion resistance. The process of fabricating vinylesters is not as complex as epoxy resins (ACI Committee 440, 2006). Vinylester resins tend to saturate the fibres more efficiently resulting in higher strength, while epoxies require much higher fibre content. Vinylesters also bond well to glass to increase the resistance of these fibres in aggressive chemical environments (Cusson & Xi, 2002).

Phenolics are a class of resins commonly used on phenol (carbolic acid) and formaldehyde and cure through a condensation reaction producing water that must be
removed during processing (ACI Committee 440, 2006). The advantages of phenolics resins include high temperature resistance, creep resistance, excellent thermal insulation and sound damping properties and corrosion resistance (ACI Committee 440, 2006). However, phenolics resins are not widely used for the construction industry. They are applied as a binder in engineered wood, brake linings, clutch plates and circuit boards.

2.5 FRP application on structural member

FRP applications have been widely used in civil engineering arena. Bank (2006) has mentioned that the used of FRP for new structural members could be divided into three; (1) FRP bars or grid reinforced concrete members, (2) FRP tendons for pre-stressed concrete members and (3) stay-in-place FRP formwork for reinforced concrete members. However, the applications of FRP for existing structural members are divided into two major types i.e. strengthening and repair. FRP strengthening means the increase of original design structure’s capacity and strength while repair is where FRP is used to retrofit an existing and deteriorated structure allowing its strength and capacity back to its original design.

FRP bars are used as an alternative material to steel reinforcement when 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). El-Sayed & Soudki (2011) highlighted the use of FRP bars as the main reinforcement for reinforced concrete members where magnetic transparency is required. Many studies on bond characteristic between FRP bars and concrete, its ductility behavior of FRP and moment distribution of members with FRP bars have also been widely covered by other researchers.

FRP tendons are used as an alternative material to steel tendons. FRP tendon has high tensile strength in the direction of the fibres and weak in the transverse direction (Elrefai et al., 2006). Zou (2003) mentioned, 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 (Youakim & Karbhari, 2006).

The use of stay-in-place FRP formwork is a system that acts to reinforce the concrete after it has been cured (Bank, 2006). FRP stay-in-place (SIP) form has been considered as an alternative to steel SIP form that 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 et al., 2008). 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).

Strengthening reinforced concrete structure using FRP material 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 a few types of strengthening using FRP which include strengthening using externally bonded FRP sheets and plates. The research on strengthening existing structure has been widely explored 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.

The strength and ductility of damaged structures of existing reinforced concrete structures due to corrosion, design failure or seismic impact can be increased through repair and rehabilitation processes using FRP. Rehabilitation of historical buildings is required because of its special characteristics and historical reasons. FRP is used because it is more suitable and superior than other types of materials as RFP is lightweight, has high strength and has better corrosion resistance.

### 2.6 Strengthening reinforced concrete structure using FRP plate

FRP composites such as FRP bar, FRP sheets as well as FRP plates have been used to strengthen reinforced concrete structures. The use of FRP plates has contributed in structural deficiencies. For example, replacement of steel reinforcement by plates may lead to corrosion due to the environment, adding tensile element of inadequate
flexural capacity of reinforced concrete using FRP plates, adding FRP plate to the tensile face to avoid of sudden failure and externally bonded stressed plates or web reinforcement to enhance shear capacity of reinforced concrete element. Hollaway & Leeming (1999) have highlighted some of the advantages using FRP plate bonding as a means for strengthening existing structures:

i) The ultimate strength of FRP plates can be varied to meet different requirement of the strengthening schemes needed.

ii) FRP plate has a density of only 20% from the density of steel and therefore the weight of FRP plate is lighter than the steel plate. This could save cost for transportation as well as installation. FRP plates do not require concrete jacket as compared to steel plate.

iii) The weight of FRP plate is low and it can be easily transferred by a single man at the site. Transporting these materials only require light vehicles without the need of lorries for delivery.

iv) FRP plate has a versatile design system compared with steel plate. The work of bonding can be done at site while steel plates are limited in length and in-situ welding may not be possible.

v) Compared to steel plates, FRP plates are more durable because they do not corrode due to the environment. The corroded steel plates could reduce the long term performance of the steel plates itself.

vi) Another advantage of FRP is due to its low conductivity of heat compared to steel, FRP plates have good resistance to fire.

vii) FRP plates do not need maintenance while steel plate will require constant monitoring and maintenance.

Despite its advantages, FRP plates also have some disadvantages. The disadvantages of FRP plates as highlighted by Hollaway & Leeming (1999) are as follows:

i) Fibre reinforced composites are an expensive material compared to steel plates. However, it is predicted that the difference of price between the two materials is expected to be reduced because of the possible increase in production.
ii) FRP plates are easily damaged by sharp objects. Compared with steel plates, the damage of FRP plates is more likely to be localized as the plate is thinner and more flexible.

2.7 Shear failure

The study on shear failure has received great attention among researchers on every aspect such as factors contributing to shear failure and design aspect. Generally, shear failure is a diagonal tension failure that is brittle in nature and should be avoided (Wang et al., 2007). In these few decades, using FRP is one of the solutions to overcome shear failure problems and since then research using FRP for shear in strengthening on structural members has been extensively conducted.

Due to the differences 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 have conducted further studies on the shear design provisions. ACI Committee 440 has 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 results from 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 is obtained.
2.7.1 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 the analysis of experimental results and model assumption rather than on the 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 et al., 1999).

Shear cracks are developed 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 (N.A) 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)
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)}

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

where:

- $Q = \text{The first moment of the area of section above the depth } y$, about the neutral axis $= Ay$
- $I = \text{Moment of inertia of the section}$

Figures 2.4 to 2.7 show 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 occur first due to flexural stresses and they start at the bottom of the beam where the flexural stresses are the largest. Next, inclined cracks occur 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 the 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)

\[ V = \frac{VQ}{Ib} \]

\[ f_i = \frac{My}{I} \]

Figure 2.5: Flexural and shear stress and principal stresses (Pillai et al., 1999)

\[ f_i = f_2 \]

\[ f_i = f_1 \]

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

Figure 2.7: Potential crack pattern (Pillai et al., 1999)
2.7.2 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 mechanisms (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). Wang et al. (2000) describe the transfer of shear in reinforced concrete without shear reinforcement by a combination of the following mechanisms:

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)

\[ \sum V = V_c + V_a + V_d \]  

(Equation 2.3)

Figure 2.8: Shear mechanism acting in a beam without stirrups (Wang et al., 2007)
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, 2003). The equation to calculate shear strength for beams without web reinforcement which is presented in ACI 318-08 (2008):

\[ V_n = V_c \]  
(Equation 2.4)

\[ V_c = \left( 1.9\sqrt{f'c} + 2500 \frac{\rho_w V_{tu} d}{M_u} \right) b_w d \leq 3.5\sqrt{f'c} b_w d \]  
(Equation 2.5)

In SI unit with \( f'c \), in MPa, ACI 318-08 gives:

\[ V_c = \left( 0.16\sqrt{f'c} + 17 \frac{\rho_w V_{tu} d}{M_u} \right) b_w d \leq 0.29\sqrt{f'c} b_w d \]  
(Equation 2.6)

The equation is then simplified where the second term will be equaled to about 0.008 \( \sqrt{f'c} \):

\[ V_c = 0.167\sqrt{f'c} b_w d \]  
(Equation 2.7)

### 2.7.3 Factors affecting the shear strength of beams without stirrups

MacGregor & Bartlett (2000) have indicated that there are five variables affecting the shear strength of a beam without stirrups:

i) **Tensile strength of concrete**

The inclined cracking load is a function of the tensile strength of concrete. A flexural crack which precedes the inclined cracks disrupts the elastic stress to a level that an inclined crack occurs at a principal tensile stress.

ii) **Longitudinal reinforcement ratio, \( \rho_w \)**

Based on the equation of \( \rho_w = \frac{A_s}{b_w d} \), when the steel reinforcement ratio is small, flexural cracks extend higher into the beam and therefore open wider than the case for large values of \( \rho_w \) and as a results, inclined cracks occur earlier.

iii) **Shear span to depth ratio, \( a_v/d \)**

Deep Beams (\( a_v/d \leq 1 \)): For deep beams, the possible modes of failure includes an anchorage failure, crushing failure at the reactions, flexural failure either from a crushing of concrete near the top of the arch or a yielding
of the tension reinforcement and a failure of the arch rib. The shear stress for deep beams has the predominant effect.

Short Beams \((1 < a_v/d \leq 2.5)\): Short beams have a shear strength that exceeds the inclined cracking strength where the load increases, after the flexural-shear crack develops, the crack extends further into the compression zone. The crack also propagates as a secondary crack towards the tension reinforcement and then progresses horizontally along the reinforcement. Failure of the beams is either by an anchorage failure at the tension reinforcement or by a crushing failure in the concrete near the compression zone.

Intermediate Length Beams \((2.5 < a_v/d \leq 6)\): For this group, the vertical flexural cracks are the first to form and followed by the inclined flexural-shear cracks. This is the usual category for beam design. The beam may fail either by shear or flexure. If shear reinforcement is not provided, the cracks propagate rapidly to the top of beam, failure occurs suddenly and termed as diagonal tension failure. If shear reinforcement is provided, load can be carried until failure occurs in shear tension mode (yielding of stirrups) or shear compression mode or flexural mode.

Long Beams \((a_v/d > 6)\): For this type of beams, the failure starts with yielding of tension reinforcement and ends by crushing of the concrete at the section of maximum bending moment. The strength of the beam depends entirely on the magnitude of the maximum bending moment and is not affected by the size of the shear force. For the design of all beams except deep beams, the shear strength is assumed to be reached when the inclined crack forms (Wang et al., 2007). Figure 2.9 shows the effect of shear span to effective depth ratio on shear strength of beam without stirrups (Sinha, 2003).
REFERENCES


ACI Committee 440 (2006). Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures. Farmington Hills, USA: American Concrete Institute


