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STATIC AND CYCLIC BEHAVIOR OF STEEL BEAMS RETROFITTED WITH FIBER REINFORCED POLYMER LAMINATES

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**SCHOOL OF
CIVIL AND ENVIRONMENTAL ENGINEERING
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SUMMARY

The rapid deterioration of aging steel infrastructures requires some significant attention in developing new techniques for effective and economical revival of these structures. External bonding of fiber reinforced polymer (FRP) laminates is a promising method. However, the up-to-date research on FRP strengthened steel beams is still very limited. It is important to understand the flexural and the bond behavior of such structures before wide application of this strengthening technique. This thesis presents a study on performance of FRP-steel joints and steel beams retrofitted with FRP under static and cyclic loading. The finding in this study could provide reference for future engineering applications of the FRP strengthening method on steel structures.

Experiments on different types of FRP-steel adhesive joints were performed. It was found that the joint with surface treated by sandblast had the highest bond strength. For joints subjected to in-plane loading, there was a critical bond length beyond with the bond strength would not increase anymore. Finite element (FE) analyses were conducted to explore the stress distribution along the bondline. The connection between adhesive and steel at the end of the bondline was the weakest link in FRP-steel adhesive joints. The stress along the bondline was highly un-uniformly distributed. The bond failure of all the joints was caused by the stress concentration at the end of bondline.

Flexural behavior of steel beams strengthened or repaired with bonded FRP laminates under static loading was experimentally investigated. It was found that the failure modes of strengthened and repaired beams were quite different. The flexural stiffness of damaged beams was increased more than that of intact beams. A simple analytical solution was developed for the flexural capacity of FRP

strengthened steel beams. The analytical and the experimental results were found to be in good agreement. FE analysis was also conducted to understand in detail stress distribution in FRP strengthened steel beams. The parametric study showed the flexural capacity of strengthened beams could almost no longer be improved when the critical bond length was reached. An optimal FRP thickness or modulus was found out before which was reached the increase of ultimate strength was more than that of flexural stiffness.

A bond failure model was proposed which can be incorporated into numerical modeling to simulate the static behavior of FRP strengthened steel beams with bond failure process. The application of this bond failure model is no longer limited by the hypothesis of elastic material or linear deformation. Geometry details at the end of bondline can also be taken into account in the model. The reliability of the model was verified by experimental results. Parametric study showed that bond strength increased with the decrease of laminate thickness, laminate modulus and adhesive thickness or with the increase of bond length.

Bond performance under cyclic loading of FRP-steel joints and FRP strengthened steel beams as well as fatigue strengthening effect of FRP laminates on damaged steel beams were explored in three series of experiments respectively. It was found that the FRP-steel joints and the strengthened beams had similar relationships between fatigue life and load level and same threshold load level. The fatigue life of damaged beams was increased more than three times by FRP. Premature bond failure did not happen until the crack in steel beam is developed large enough. The FE result showed the stress concentration level can be effectively reduced by FRP.

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LIST OF SYMBOLS

b_0	Bond width
b_f	Flange width
b'	Width of virtual section
d	Beam depth
d_{ce}	Distance from elastic compressive force to neutral axis
d_{cp}	Distance from plastic compressive force to neutral axis
d_f	Distance from tensile force in FRP laminate to neutral axis
d_{te}	Distance from elastic tensile force to neutral axis
d_{tp}	Distance from plastic tensile force to neutral axis
D	Mid-span deflection
D_0	Mid-span deflection at bond failure
D_e	Experimental mid-span deflection
D_f	Numerical mid-span deflection
D_N	Normalized mid-span deflection
E	Young's modulus of steel
E_c	Young's modulus of CFRP laminate
E_f	Young's modulus of FRP laminate
E_a	Young's modulus of adhesive
E'	Young's modulus of transformed section
f_a	Tensile strength of adhesive
f_f	Tensile strength of FRP laminate
f_y	Yield strength of steel
F_c	Compressive force in steel section
F_{ce}	Elastic compressive force in steel section
F_{cp}	Plastic compressive force in steel section
F_f	Tensile force in FRP laminate
F_t	Tensile force in steel section
F_{te}	Elastic tensile force in steel section

F_{tp}	Plastic tensile force in steel section
G_I	Energy release rate under mode one loading
G_{II}	Energy release rate under mode two loading
G_{IC}	Critical energy release rate under mode one loading
G_{IIC}	Critical energy release rate under mode two loading
h	Depth between fillets
I	Area moment of inertia
I'	Area moment of inertia of strengthened section
I_f	Area moment of inertia of flange
I_w	Area moment of inertia of web
I'_w	Area moment of inertia of web of transformed section
K	Bulk modulus
l	Beam length
l_o	Bond length
l_{oN}	Normalized bond length
l_1	Distance between loading points
l_2	Distance between support and loading point
l_c	Crack length
L	Span of beam
m	Shape parameter
M	Bending moment
M_e	Maximum elastic bending moment
M_e'	Maximum elastic bending moment of strengthened section
n_e	Number of elements
N	Number of cycles
N_f	Number of cycles to failure
N_l	Number of laminate layers
N_N	Normalized number of cycles to failure
p	Pressure
P	Ultimate load
P_0	Reference load

P_N	Normalized load
R	Outer radius of tube
s	Stiffness
s'	Stiffness of strengthened section
S	Bond strength
t	Thickness of steel plate
t_w	Thickness of web
t'_w	Thickness of transformed web
t_a	Thickness of adhesive
t_0	Thickness of FRP laminate
t_f	Thickness of flange
u	Displacement in numerical model
u_0	Initial displacement in numerical model
u_c	Critical displacement in numerical model
u_s	Displacement in numerical submodel
u_{s0}	Initial displacement in numerical submodel
u_{sc}	Critical displacement in numerical submodel
v	Volume of material
v_i	Volume of element
V	Shear strength
V_{ave}	Average shear strength
w	Strain energy density
w_d	Distortional strain energy density
w_{dc}	Critical distortional strain energy density
w_e	Equivalent strain energy density
w_{ec}	Critical equivalent strain energy density
w_{el}	Equivalent strain energy density of reference specimen
w_{en}	Normalized equivalent strain energy density
w_r	Representative strain energy density
w_{r0}	Characteristic strain energy density
w_v	Dilatational strain energy density

w_{vc}	Critical dilatational strain energy density
x	Distance bondline end
y	Distance to neutral axis
y_0	Distance from bottom extreme fiber neutral axis
y_l	Distance from neutral axis to bottom of plastic zone
y_p	Depth of plastic zone
μ	Poisson's ratio
σ	Stress
σ'	Stress of transformed section
σ_0	Scale parameter
σ_e	Equivalent interfacial stress
σ_i	Principal stress tensor
σ_{ij}	Stress tensor
σ_s	Shear stress
σ_t	Tensile stress
σ_{th}	Threshold stress
σ_y	Yield stress of steel
σ'_y	Yield stress of transformed section
σ_u	Ultimate stress of steel
σ'_u	Ultimate stress of transformed section
ε_0	Strain in extreme bottom fiber of beam
ε_c	Compressive strain
ε_i	Principal strain tensor
ε_{ij}	Strain tensor
ε_{max}	Maximum strain in FRP laminate
ε_t	Tensile strain
ε_y	Yield strain
ε'_y	Yield strain of transformed section
θ	Unit volume change
λ_I	Mode one parameter
λ_{II}	Mode two parameter

λ_d	Distortional correlative parameter
λ_v	Dilatational correlative parameter
π	Ratio of circumference to diameter
ρ	Radius of curvature
Δ	Relative increase

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

A large number of steel infrastructures are either structurally or functionally deficient after many years of service. For instance, in the United States, roughly 15% of bridges are structurally deficient (FHWA, 2001). Of these deficient bridges, 56% have steel superstructures. Causes of the deterioration can be attributed to the age of the structure, steady increase in the weight of highway vehicles and deterioration due to environmental attack. In addition to deterioration, some of these bridges have to be upgraded to meet the present and future traffic needs. Traditional retrofit methods such as attaching steel plates to the original members have several disadvantages, such as difficulty of installing heavy steel plates, increased self weight, weld-induced residual stress, and the potential for weld fatigue cracking at the cover plate ends, etc. Consequently, novel yet viable and cost-effective strengthening techniques are still very necessary for such steel structures.

The advanced composite material fiber-reinforced polymer (FRP) has experienced a continuous surge in structural engineering applications around the world during the last decade. The desirable characteristics of FRP such as high strength, high tensile modulus, light weight, installation facility and resistance to corrosion have all played a very significant role in creating greater interest for structural repair and strengthening applications. Strengthening in-service steel beams through the use of externally bonded composite plates is particularly attractive due to the ease of application. The FRP material weighs approximately one-tenth of steel. It can be adhesively bonded and has stiffness comparable to that of steel in addition to being

corrosion resistant. Due to these excellent properties of FRP, it has tremendous structural applications in retrofitting of steel bridges, especially in strengthening steel beams and other flexural members.

1.2 OBJECTIVES

While extensive studies have been conducted on the repair and strengthening of concrete structures using FRP materials, only a very limited amount of research has been carried out on the application of these materials to strengthening steel structures. The mechanical property of steel is totally different to that of concrete, hence, the mechanical behavior of structures made from these two materials may differ greatly when strengthened or repaired with the FRP materials.

The main objectives of the current research are:

- (a) to investigate the feasibility and the effectiveness of strengthening steel bridge beams through external bonding of FRP laminates by extensive studies of strength, stiffness and mechanism of FRP strengthened steel beams;
- (b) to explore in detail bond performance of FRP-steel joint, debonding mechanism of FRP strengthened steel beams and parameters influencing bond strength and to develop a bond failure model which can be used to predict bond failure load;
- (c) to understand cyclic behavior FRP strengthened steel beams, fatigue resistance capacity of FRP-steel joint and strengthening effect of FRP on damaged steel beams under cyclic loading;

The reasons to have the above research objectives are as follows:

- (a) The fundamental flexural behavior of the FRP strengthened steel beams must be clearly understood before other investigations, so that the static experiments are to be conducted first.
- (b) With the understanding of the flexural performance and the strengthening effect, the bond failure of the strengthening structures must be investigated since the bond failure mode of FRP-steel structures is quite different from the extensively explored failure of FRP-concrete structures. And actually it is a vital issue of the successful application of this strengthening method.
- (c) In practice, numerous steel structures are subject cyclic loading in service. Therefore, besides static behavior, the performance of the strengthened steel structures under cyclic loading and the effect of fatigue life extension should also be investigated.

1.3 SCOPE AND PLAN OF CURRENT RESEARCH

To achieve the proposed objectives, various research activities including experiments, numerical and theoretical analyses are involved. The scope and plan of the current research can be summarized as follows:

- (a) Experiments on steel beams strengthened with FRP laminates with different configurations under static loading are to be conducted, since experiment is the most intuitive method to investigate the strengthening effect of bonded FRP laminates. The flexural performance and the failure mode of both strengthened and un-strengthened beams can be directly observed from the experimental program. To understand in detail the stress distribution and the mechanism of the FRP strengthening, numerical analysis is to be carried out. The experimental results are to be used to verify the validity of the numerical model. Then the influence of more strengthening parameters on strengthening effect can be numerically investigated since there is almost no

economic limitation as that on the experimental investigation. Thanks to the solvability of the load and the deflection without consideration of bond failure, the analytical solution of the flexural capacity of FRP strengthened steel beams is to be developed.

- (b) Bond performance of the FRP-steel structure is to be studied extensively since it is a very importance issue for FRP strengthened steel structures. Since the bond strength of FRP-steel joint can not be calculated or estimated based on the material and geometry properties, experimental study must be carried out first. Experiments on different types of adhesively bonded FRP-steel joints with different configurations are to be conducted to have a fundamental understanding of the bond behavior. The stress distribution along the bondline is to be numerically investigated since it is too complicated to be studied directly by means of experiment. The mechanism of the bond failure of FRP-steel joint can be investigated by virtue of the detailed numerical analyses. Based on the experimental results of the FRP-steel joints and the FRP strengthened steel beams with bond failure, a bond failure model to predict the premature debonding of FRP laminates from strengthened steel beam is to be developed. With the validity of the model verified by the experimental data, more numerical analyses are to be conducted to understand the influence of different strengthening parameters on the bond failure load of FRP strengthened steel beams.
- (c) Cyclic behavior of FRP strengthened steel beams is to be investigated to understand the strengthening effect of FRP on steel beams comprehensively. Two issues of FRP strengthened steel beams under cyclic loading are to be studied, i.e. bond performance under cyclic loading and effect of fatigue life enhancement of steel beams by FRP strengthening. Experiments on FRP-steel joints and FRP strengthened steel beams are to be conducted to find out the relationships between fatigue bond strength and stress level

under different load amplitude. Based on this relationship the bond failure of FRP strengthened steel structures can be effectively prevented. The strengthening effect of FRP on fatigue resistance of damaged steel beams is also to be experimentally studied. FRP strengthened steel beams with different strengthening parameters tested under constant cyclic amplitude are to be investigated to evaluate the effect of the FRP laminate on fatigue life extension.

1.4 ORIGINALITY OF CURRENT RESEARCH

While many studies have been conducted on the repair and strengthening of concrete structures using advanced composites, only a very limited amount of research work have been conducted on the application of these materials to steel structures.

According to the report of Buyukozturk et al. (2004) together with the author's knowledge, documented research results on cyclic behavior of FRP strengthened steel beams are very limited, although some experimental studies on static performance have been carried out (Miller et al., 2001; Sen et al., 2001; Al-Saidy, 2004).

Even for static behavior of steel beams strengthened with FRP, previous research was mainly limited to experimental investigations. Although Deng et al. (2004) had performed numerical analysis on the stress distribution in the bondline of FRP strengthened steel beams, no numerical analysis with incorporation of bond failure model can be found in published literatures to the date when this study was initiated.

With this background, the essential originality of this research work can be summarized as follows:

- (a) Flexural behavior of FRP strengthened steel beams with different combinations of strengthening parameters was explored. Although some experimental studies of flexural performance of FRP bonded steel beams have been carried out by other researchers, a systematic study of various strength-influenced parameters such as, laminate thickness, bond length and laminate stiffness has not been performed and comprehensively understood yet. Based on the experimental investigation, analytical solution for flexural capacity of FRP strengthened steel beams was developed. Numerical modeling was also performed to study the stress distribution and effect of more parameters was explored.
- (b) A bond failure model which can be used to predict the bond failure of FRP strengthened steel beams was proposed. This model can be incorporated into numerical analysis to simulate the strengthened beams with bond failure process. The dominant failure mode for FRP strengthened steel beams is adhesive failure rather than adherend failure. This is the major difference compared to the failure mode of FRP retrofitted concrete structures. Due to the complexity of geometry and material property of the bi-material adhesive joint, there is no uniform failure criterion for FRP-steel structures. Based on experiments on various types of FRP-steel adhesive joints, stress and strain distributions in the bondline were studied in detail. Original bond failure model employing strain energy density as failure criterion was proposed in this study. It can be incorporated into numerical modeling to predict the bond failure. If premature bond failure is not considered in numerical modeling, the failure load from FE analysis will be much higher than that from experiments.

(c) Cyclic behavior of FRP strengthened steel beams and adhesively bonded FRP-steel joints was experimentally investigated. Although the FRP strengthening system is likely to perform satisfactorily under low-amplitude loading as discussed by Buyukoztuerk et al. (2004), the performance of such systems under high-amplitude cyclic loading conditions is still unknown despite its critical importance in bridge applications. The strengthening effect of FRP laminates on damaged steel beams under high amplitude cyclic loading was investigated in this study. Bond performance under cyclic loading was also studied. The fatigue life of steel beam was found to be enhanced up to three times and the fatigue threshold of the bond strength was estimated.

1.5 TERMINOLOGY

Throughout the report, several terminologies are mentioned which require to be defined clearly. They are listed as follows:

Adherend — body that is or intended to be held to another body by an adhesive.

Adhesive — a substance capable of holding materials together by surface attachment.

Adhesive joint — location at which two adherends are held together with a layer of adhesive.

Balanced joint — an adhesive joint with identical upper and lower adherends, i.e. the adherends have same materials, geometries and dimensions. (Antonym for *unbalanced joint*).

Bond failure — separation of a bonded joint or interface. (Synonym for *debonding*).

Bondline — the layer of adhesive which attaches two adherends. (Synonym for *glue line*).

Carbon fiber reinforced polymer (CFRP) — a fiber-polymer composite that contains carbon fibers as reinforcing material.

Fiber-reinforced polymer (FRP) — a polymer that contains fibers in various forms such as cloth, mat or strands to enhance properties.

Glass fiber reinforced polymer (GFRP) — a fiber-polymer composite that contains glass fibers as reinforcing material.

Retrofit — add machinery or equipment to an existing piece of equipment or system to correct a defect or add capability. (Synonym for *strengthen*).

:

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION OF COMPOSITE MATERIAL

2.1.1 Definition of FRP

FRP is a composite material which is primarily composed of reinforcement fiber and polymer matrix. Polymer matrix is commonly made of thermosetting resins, such as polyester, urethane methacrylate, vinylester, epoxy and phenolic. They are isotropic materials which allow load transfer between the fibers, but they also perform several other duties. The matrix protects notch-sensitive fibers from abrasion and it forms a protective barrier between the fibers and the environment, thus preventing attack from moisture, chemicals and oxidation. It also plays an important role in providing shear, transverse tensile and compression properties. In Figure 2.1 (Tok, 2003), the configurations of common forms of fiber reinforced polymers are illustrated.

Overall, the properties of the composite are determined by:

- the properties of the fiber,
- the properties of the resin,
- the ratio of fiber to resin in the composite (Fiber Volume Fraction), and
- the geometry and orientation of the fibers in the composite.

The illustration of the combined effect on modulus of the addition of fibers to a resin matrix is shown in Figure 2.2 (Azom, 2004).

2.1.2 Classification of FRP

According to the types of reinforcement fibers, FRP materials fall into three main families of glass, carbon and aramid (Cogswell, 1992):

(a) *Glass fibers* are the most widely used and the cheapest reinforcement. It is available in three main types. E-Glass is the standard form and is also known as electrical grade. It has very low alkali content and good electrical, mechanical and chemical properties. C-Glass is a chemical resistant grade and is often used where protection from corrosive environment is required. S-Glass, also referred to as R-Glass, has enhanced mechanical properties and is mainly used for aerospace applications.

(b) *Carbon fibers* are produced by the controlled oxidation and carbonisation of cellulose, pitch or polyacrylonitrile materials, known as precursors. The main impetus for the development of carbon fibers came from the aerospace industry with its need for a material with a combination of high strength, high stiffness and low weight (Chung, 1994). Like glass, carbon fiber can come in various forms such as woven material, chopped strand and continuous filament. However, unlike other fibers, carbon has a slightly negative coefficient of thermal expansion along the fiber axis. This means that composites with a longitudinal coefficient of thermal expansion of zero can be produced with carbon fiber. They have the limitation of poor impact resistance.

(c) *Aramid fibers* belong to a class of materials known as liquid crystalline polymers. They have a superior strength to weight ratio when compared to glass fibers and also provide excellent abrasion resistance in a composite. They are the fibers of choice in bullet-proof vests. However they are poor in compression,

typically offering 1/3 of their tensile performance. Hence, the use of aramid fibers in applications that are subject to high strain compressive or flexural loads is limited.

The comparison of mechanical properties of those fibers is shown in Table 2.1 (Peters, 1998). It can be seen clearly that, of all these composite materials, carbon fiber has the largest values of the essential mechanical properties, i.e. tensile modulus and tensile strength.

2.1.3 Advantages of FRP in Civil Infrastructural Applications

The property combined from fibers and matrix potentially gives the designer a wide palette of material choices to fit the specific requirements of the structure and shows immense potential to add to the current palette of materials being used in civil infrastructure. The comparison of mechanical properties of FRP and other common structural materials is shown in Figure 2.3 and 2.4 (Buyukoztuerk et al., 2004).

Some of the important advantages (Karbhari and Zhao, 2000) of FRP in civil infrastructure applications are listed below:

- (a) High specific strength and stiffness
- (b) Enhanced fatigue life
- (c) Corrosion resistance
- (d) Controllable thermal properties
- (e) Tailored properties
- (f) Non-magnetic properties

2.2 APPLICATION OF FRP IN STRUCTURAL REHABILITATION

2.2.1 FRP Retrofitted Concrete Structures

External steel plate bonding has been applied to the retrofitting of reinforced concrete slabs and beams extensively in the past (McKenna and Erki, 1994; Uy et al., 2002). The benefits of CFRP materials have also been applied to similar applications due to their high stiffness-to-weight and strength-to-weight ratios of these materials combined with their superior environmental durability, which have made them a competing and attractive alternative to the conventional strengthening and repair materials. In recent years FRP materials have experienced a continuous increase in usage for structural strengthening and repair applications around the world. It has been shown through experimental and theoretical studies that externally bonded FRP composites can be used to improve the desired performance of a concrete structural member such as load carrying capacity and stiffness, ductility, performance under cyclic and fatigue loading and environmental durability (Teng et al., 2002; Bakis et al., 2002; Eide et al., 2003; Oehlers and Seracino, 2004).

Strengthening with FRP composites can be applied to various types of structural members including beams, columns, slabs and walls. Depending on member type, the objective of strengthening may be one or a combination of several of the following:

- (a) to increase axial (Chaallal et al., 2003), flexural (Teng et al., 2001; El-Hacha et al., 2004) or shear (Oehlers and Seracino, 2004) load capacities;
- (b) to increase ductility for improved seismic performance (Mo et al., 2004);
- (c) to increase the remaining fatigue life (Heffernan and Erki, 2004);
- (d) to increase durability against environmental effects (Gheorghiu et al., 2004).

2.2.2 FRP in Other Structures

Wood structures:

In order to avoid cracks on the timber walls, Premrov et al. (2004) strengthened the wood walls with CFRP strips. Nine samples were tested. It was shown that the inclusion of CFRP diagonal strip reinforcement on the load-carrying capacity could be quite high and that it was maximized when the carbon strips were connected to the timber frame. On the other hand, the ductility itself was not significantly improved.

A wood pile repair method that utilizes FRP composite shells and a grouting material was proposed (Lopez-Anido et al., 2005). The proposed repair of wood piles with prefabricated FRP composite has a dual function of marine borer protection and structural restoration.

Aluminum structures:

Broughton et al. (1997) discussed the flexural behavior of compound beams in which unidirectional CFRP was externally bonded to extruded aluminum box-sections. Four unreinforced and four reinforced beams were tested. Close correlation between theory and experiment was observed and maximum gains in stiffness and ultimate strengths of 75% and 63%, respectively, were demonstrated with only a 7% weight increase.

Pantelides et al. (2003) investigated a repair method for cracked aluminum welded connections of overhead sign structures by externally wrapping GFRP composites. By static tests, it was demonstrated that capacities from 1.17 to 1.25 times that of the intact connection could be reached by strengthening the damaged connection

with crack length ranged from 24% to 66% of total weld length. By providing adequate surface preparation and bond length, the tack-welded connections with GFRP composites reached capacities of 95% to 99% of welded connection with no visible cracks.

FRP strengthened FRP structures:

Nordina and Täljsten (2004) investigated the flexural capacity of composite GFRP beams strengthened by bonding of CFRP sheets. Three beams were tested under four-point bending load with total length of three meters. One beam consisted of a glass fiber I-beam with carbon fiber strengthened bottom flange. The other two had a rectangular concrete block in the compressive zone. The tests showed that a GFRP girder strengthened with CFRP could be a promising load-carrying element in a short-time perspective. The low weight combined with high strength is an excellent alternative to traditional steel and concrete beams. It was, however, important to understand the materials and their strengths and weaknesses, for example the stability issue was of tremendous importance.

2.3 FRP RETROFITTED STEEL STRUCTURES

2.3.1 Rehabilitation of Deteriorated Steel Beams

A series of experimental tests on the stiffness and strength of steel girders strengthened with composite materials was conducted by Gillespie et al. (1996, 1997). Four full-scale girders were removed from an old deteriorated steel bridge. All girders had corrosion along their length and the corrosion was mostly concentrated on the tension flange. The evaluation of the girders indicated approximately 40 percent loss of the tension flange. In the experimental program

only the bottom flanges were reinforced, along the entire length of the girders, with a single layer of CFRP strips. It was found that among various reinforcing geometries and schemes, reinforcement using pultruded strips resulted in the largest increases in stiffness and strength. Rehabilitation of four severely corroded steel girders using pultruded CFRP strips yielded girders with 97% of their original stiffness and an ultimate capacity that was 113% of the original section strength, indicating the efficiency of the rehabilitation method.

By using CFRP strips, Miller et al. (2001) strengthened a bridge made of standard steel I-beams. Strengthening was completed using 5.25 mm thick strips, resulting in a stiffness increase of twelve percent, based on vehicle load tests before and after the strengthening. Using the method of transformed sections, a strain decrease of ten percent was predicted. It was noted that no particular amount of strengthening was required and the project requirements were directed towards providing long-term performance and durability data.

2.3.2 Retrofit of Artificially Degraded Steel Beams

Research programs on the feasibility and effectiveness of using FRP to strengthen artificially degraded steel beams were carried out by several parties in recent years. For instance, Sen et al. (2001) conducted a study on CFRP strengthened steel girders with a concrete slab. The stiffness and strength of the specimen were compared before and after rehabilitation with CFRP laminates that were bonded to the outer surface of the tension flange. Although the stiffness of the girders was not increased significantly, the strength of the rehabilitated girder was increased from 9% to 52%.

Liu et al. (2001) discussed a strengthening method using FRP material for repairing defected steel members by performing a series of tests on four W12×14 girders. A 102 mm wide notch was imposed in the tension flange of each beam to simulate the severe loss of section due to corrosion. An increase in the strength and stiffness compared to the notched beam without strengthening was observed, although the strength of the original section was not completely restored. The maximum increase in the plastic load capacities was found to be 60%.

Tavakkolizadeh, and Saadatmanesh (2001) examined eight S5x10 steel beams, 1300 mm long, under a four-point bending configuration. Six specimens were notched in the middle of the tension flange to depths of 3.2 mm and 6.4 mm. The results indicated that stiffness and ultimate load carrying capacity of the repaired beams were close to their intact values by bonding the 0.13 mm thick CFRP sheets, regardless of the length of the patch. Significant increase of the ultimate load carrying capacity was recorded, as high as 63 and 144 percent for shallow and deep notches, respectively. Tavakkolizadeh and Saadatmanesh (2003a) also tested a total of three large-scale composite girders. The tension flanges of the girders were cut with different depths to simulate 25%, 50% and 100% loss of tension flange. It was demonstrated that a considerable increase of static strength and stiffness of steel structures could be obtained.

Al-Saidy et al. (2004) performed tests on steel-concrete composite beams that had a portion of the flange removed to simulate corrosion damage. Two different levels of damage were simulated to represent 50 percent and 75 percent loss of the area of the tension flange. Three of the beams were repaired using two different repair schemes. In the first repair scheme CFRP laminates were bonded to the bottom portion of the web of the beam. In the second scheme the laminates were bonded to the web and the remaining portion of the tension flange of the beam. Strength

increases ranging between 6 percent and 24 percent as compared to the undamaged beams were reported. While the capacity of the beams could be restored to levels higher than those for the undamaged beams, only fifty percent of the lost stiffness was recovered.

2.3.3 Strengthening Undamaged Steel Beams

For steel beams without initial damage, the effect of strength improvement by bonding FRP laminates was investigated as early as 1996 in Delaware University (Edberg et al., 1996; Gillespie et al., 1996). The beams were tested with a span of 1.37 m in four-point bending. The beams were loaded and unloaded within the elastic region prior to failure and only one of the strengthening systems exhibited any substantial deterioration of stiffness indicating a deterioration of the bond between the steel and the CFRP. Prior to testing the beams to failure a steel plate was bolted to the top flange of the beams to prevent buckling of the compression flange. Application of the different strengthening configurations increased the elastic stiffness in the range of ten to thirty percent while the yield load increased by thirty-seven to seventy percent.

GFRP material was also used to strengthen steel beams under flexural loading (Abushagur, 2003). Steel W150 x 37 beams were strengthened by bonding 19 mm thick GFRP plates to the top and bottom flanges, maintaining a symmetric cross-section. Test results indicated that an initial stiffness increase of fifteen percent could be achieved, with an increase in the yield load of twenty-three percent. Failure was by delamination between the plies of the GFRP laminate on the tension side of the beam. At ultimate, the strength of the beam was increased by 78 percent. Despite the fact that the beams were heavily reinforced, little improvement

in the stiffness before yield was noted due to the low elastic modulus of this GFRP material.

Tavakkolizadeh, and Saadatmanesh (2003b) carried out experimental studies on three composite girders with W14x30 steel beams and 75 mm thick and 910 mm wide concrete slabs. The three specimens were strengthened with 1, 3, and 5 layers of CFRP sheets, respectively. Four-point bending tests were then performed on the 4780 mm long specimens. The observed modes of failure for the beams with 1, 3, and 5 layers were concrete slab crushing, debonding of FRP, and concrete slab crushing combined with failure in the web respectively. A significant increase of the ultimate load-carrying capacity of the girders was achieved. The recorded increase was 44, 51, and 76 percent for the beams retrofitted with 1, 3, and 5 layers of sheets, respectively.

Patnaik and Bauer (2004) tested three full-scale built-up wide-flange beams in four-point bending with a span of 3150 mm. Two of the beams were strengthened by adhesively bonding full-length standard modulus CFRP laminate strips across the entire width of the bottom of the tension flange of the beams. No debonding was noted until after rupture of the CFRP strips. The reported plastic moment capacity of both strengthened beams was approximately 14% greater than that of the unstrengthened beam. Since there was no stiffener installed, both beams failed by lateral-torsional buckling.

Deng et al. (2004) investigated stress distribution in the interface between CFRP plate and steel beam. A simple closed-form solution to calculate the interfacial stresses of retrofitted steel beams strengthened with CFRP plates under thermal and mechanical loads was presented. The tapers were found to reduce about the

maximum shear stresses by about 30% and the maximum normal stresses by about 50%.

2.3.4 Strengthening Other Steel Structures

Vatovec et al. (2002) investigated the strengthening of steel tubes in flexure by bonding CFRP strips to the extreme bending faces. They found that the addition of the CFRP strips provided limited increase in the ultimate capacity of between 6% and 26%, depending on the geometry of the rehabilitation. The strengthening was limited by debonding of the CFRP. The retrofitted beams exhibited ductile behavior near ultimate load.

Jiao and Zhao (2004) investigated the tensile strength of butt-welded very high strength (VHS) circular tubes strengthened with CFRP by an experimental study of eight CFRP wrapped VHS tubes. Six plies of carbon fiber sheets were bonded to the tubes with different bond lengths. It was found that an increase of strength varying from 25% to 76% was obtained by this strengthening method. The effective bond length of 150 mm for CFRP bonded VHS tube was found out.

To investigate the effect of bonded CFRP on web crippling capacity of cold-formed rectangular hollow section (RHS), Zhao et al. (2006) conducted test on CFRP strengthened RHS. Several strengthening methods including wrapping CFRP sheets outside the RHS, or applying CFRP plates outside or/and inside the RHS were adopted. CFRP strengthening was found to be able to significantly increase the web crippling capacity, especially for those with large web depth-to-thickness ratios. The authors believed that there were three main reasons for improved behavior, i.e. the change of failure mode from web buckling to web yielding, increased restraints against web rotation and achievement of material strain hardening.

Teng and Hu (2004) conducted axial compression tests on GFRP strengthened steel tubes. Continuous glass fiber sheet were wrapped around the steel tubes to form a jacket with the finishing end of the fiber sheet overlapping its starting end by 150 mm to ensure circumferential continuity. The test result showed that FRP jacketing was a very promising technique for the retrofit and strengthening of circular hollow steel tubes under compression. The ultimate load of the steel tube was enhanced from 5% to 10% by FRP jackets of different thicknesses. The ultimate load increased with the thickness of the FRP jacket, although this increase was generally very limited. The axial shortening at peak load was enhanced by around ten times through FRP confinement. FRP jacketing was also believed to be a very promising technique for the seismic retrofit of circular steel tubular columns.

The strengthening effect of CFRP on fire-exposed concrete-filled steel tubular (CFST) beam columns was explored by Tao and Han (2007). The jacketing of carbon fiber sheets was applied using a hand lay-up method. The finishing end of a sheet overlapped the starting end by 150 mm. The specimens were tested as pin-ended supported and subjected to single curvature bending. The test results showed that the load-bearing capacity of the fire-exposed CFST beam columns was enhanced by the CFRP jackets to some extent, while the influence of CFRP repair on stiffness was not apparent. The strength enhancement from CFRP confinement decreased with the increasing of eccentricity or slenderness ratio. Ductility enhancement was also observed except those axially loaded shorter specimens with rupture of CFRP jackets at the mid-height, occurred near the peak loads.

2.3.5 Applications

Lane and Ward (2000) discussed the successful restoration of the Tickford bridge, which was built up in 1810 as shown in Figure 2.5(a). A continuous filament

polyester drape veil was installed to provide insulation between the carbon fiber and the bridge girder. Carbon fiber prepreg sheet was applied in up to 14 layers and repainted. The retrofit was made unnoticeable by employing composites with same color as the existing structure. Final tests confirmed that the strengthening had upgraded the capacity of the bridge to 40 ton vehicles.

Miller et al. (2001) reported that the upgrading of the Christina Creek Bridge using CFRP patches. A set of six pultruded CFRP patches, 1.5m length, was bonded in parallel to the bottom side of the tension flanges. In order to ensure a perfect match of the patches, a staggered joint was created. An 11.6% increase in stiffness and 10% decrease of deformation were achieved.

Garden (2001) described the repair of steel I-beams on the Boots Building in Nottingham. The composite comprised unidirectional, $0^\circ/90^\circ$, and $\pm 45^\circ$ fiber orientations. The unidirectional fibers were aligned along the direction of the beam's length for flexural strengthening. The $0^\circ/90^\circ$ and $\pm 45^\circ$ fiber directions were used to resist shear and torsional loading. Figure 2.5(c) illustrates the final placement of the carbon fiber prepreg layers around the flanges and web of the steel beam.

The retrofit of Hythe Bridge, which was built in 1874 over the River Thames, was presented by Luke (2001). The bridge was made up of eight cast iron T-girders with a clear span equal to 7.8 m. The main girders were strengthened with prestressed pultruded laminates and CFRP sheets as shown in Figure 2.5(d). After strengthening, there was no tensile stress observed in the girders under a live load up to 400 kN.

Moy and Bloodworth (2007) discussed the rehabilitation of steel girders of the Acton Bridge in London. Pultruded CFRP plates were applied to the tension flanges of the girders and then fixed with clamps. Monitoring of the girders under service conditions confirmed effectiveness of the strengthening in reducing the stress level by 25%.

From the research investigations on FRP retrofitted steel beams, it can be argued that:

- (a) Some qualitative studies have been performed on investigating static behavior of FRP repaired steel beams, but mainly they are limited to experimental studies. Very limited publication on detailed stress distribution, bond failure mechanism and numerical modeling of FRP retrofitted steel beams can be found in the literature.
- (b) Although the research on fatigue-related behavior of FRP-steel beams has been carried out recently, no research on high-amplitude cyclic or seismic behavior of FRP retrofitted steel beams can be found in any published paper so far.

2.4 BOND BEHAVIOR

For FRP-steel adhesive joints, adhesive failure is the dominant failure mode, although other failure modes are theoretically possible (Figure 2.6). Therefore, adhesive failure criteria are of great significance for investigation of the FRP-bonded steel structures.

Bond failure behavior has been explored and different failure criteria for adhesive joints have been developed in the literature (Adams et al., 1997; Tong and Steven, 1999). According to different mechanisms, these failure criteria can be subdivided

into three categories: (a) point-based criteria; (b) zone criteria (c) fracture mechanics.

2.4.1 Point-Based Criteria

For this type of failure criteria, adhesive failure is assumed to occur when a critical value of stress or strain or a combination of components (effective value) with appropriate weighting factors attains its limiting value. An application of the effective value is shown in Figure 2.7 (Lee and Kong, 2000). One of the best-known failure criteria is the Von Mises criterion. This type of failure criterion has been used quite extensively in finite element analyses, and it is one of the standard failure criteria in commercial software. The criterion can easily be applied in conjunction with a geometric and material nonlinear finite element analysis to perform failure analysis of adhesively bonded joints. It neglects, however, the hydrostatic component of stress, which can significantly affect the behavior of polymers. Other proposed criteria include maximum shear, peel or principal stress criterion. Throughout the literature, these criteria have also been applied in strain-based form.

Special attention has to be paid to the fact that these criteria can only be used to determine the adhesive strength if the maximum fillet stress or strain is not present at a point of singularity. For most joints (e.g. single-lap, double-lap) this will not be the case. The stress and strain distributions in adhesive are shown in Figure 2.8 and 2.9 (Hart-Smith, 1973). Two issues may arise if singularity points are not considered: first, the numerical value is physically not relevant at the point of singularity as it approaches infinity; and failure initiation may be predicted at a point of singularity rather than the real failure location as discussed by Clark and McGregor (1993). Alternative approaches with point-based criteria determine the

maximum value for a given mesh size which is a common way to scale mesh dependent results (McCarthy, 1999). This method introduces the element size as a new variable length scale to the problem, which does not have any real physical meaning, but most likely depends on the geometry of the studied problem. This approach can therefore work for one type of joint, but has to be calibrated repeatedly when changing the joint geometry.

2.4.2 Zone Criteria

Zone-based methods introduce the size of a damage zone as a new length scale into the problem. Nevertheless, the dependency of failure on a damage zone is physically reasonable and can be explained with damage evolution in the underlying microstructure. Two different approaches can be distinguished: some researchers propose integration of an appropriate failure measure (e.g. Von Mises stress or strain, principal stress or strain) over a certain zone to obtain the critical failure value (average zone criterion), while others determine a critical zone where the failure measure exceeds the respective failure value everywhere within its boundaries (critical zone criterion). The latter method does not require any integration of the failure measure, and established zone values will generally be smaller than for the averaging method.

An average zone criterion has been used in the literature for composites (Whitney and Nuismer, 1974; Joo and Sun, 1992) to overcome the problem of failure stresses at singularity points. Soutis et al. (1999) adopted this criterion to predict adhesive failure. It is based on the idea that the actual stress field in the structure does not follow a singularity function, as local plastic deformation causes stress redistribution. This is especially true for adhesives with high ductility.

A critical zone criterion was proposed for either distances (Whitney and Nuismer, 1974; Crocombe et al., 1990; Towse et al., 1999) or areas (Clark and McGregor, 1993; Sheppard et al., 1998). This criterion states that the material will fail once an appropriate failure measure exceeds the ultimate strength of the material everywhere within a critical distance or zone (Figure 2.10). The damage occurs because the material is locally subjected to strains or stresses greater than the ultimate material properties. Fracture does not occur by the propagation of one single, sharp crack, but rather by the development and propagation of a damage zone containing micro cracks and voids. Voids and flaws will be initiated on a micro scale within the adhesive. Under medium load, these damage zones will grow in size and the concentration points of specific damage will increase. As the failure load is reached, the damage zone will grow to a critical size when the individual components of damage will coalesce and finally form an unstable crack (Sheppard et al., 1998). Brittle adhesives will fail catastrophically once the critical zone size is reached (Papini et al., 1994).

2.4.3 Fracture Mechanics

This approach recognizes that all materials contain flaws and that adhesive joints usually fail by the initiation (or subcritical growth) and propagation of flaws of a critical size within the adhesive layer. Fracture mechanics attempts to link these processes with predictions of the joint strength under various loading conditions by analyzing the stress state within the joint, particularly in the vicinity of the critical flaw and comparing this with material properties that describe the tendency of the cracks to propagate.

The method was first applied to polymers and adhesive bonds by Kinloch (1987). A fundamental tenet of fracture mechanics is that the fracture toughness K_C (or critical

stress intensity factor) and rate of change of potential energy with crack area J_C (or strain energy release rate G_C for linear elastic fracture) are material properties. Considerable effort has therefore gone into the measurement of these properties in both bulk specimens (Davidson, 1995) and adhesive joints. Within the joint, the high stiffness adherends constrain the crack propagation to run along the joint, driven by the peel stresses - present even in so called 'shear' joints - for mode I fracture and shear stresses for mode II fracture (Kinloch, 1987). For prediction of crack growth and failure, analysis of the stresses in the vicinity of the crack is required usually requiring detailed finite element analysis because of the high stress gradients encountered within the joint (Crompton and Clark, 1988; Groth, 1988).

A few limitations arise at this point due to the constraining effect of the substrates. First, G_C is generally regarded as material property, which depends on the plastic zone and the fracture mode developing in the material. Clearly, in a thin bondline, this zone will be restricted, which introduces geometry dependence for the critical energy release rate. Second, the fracture mode might be influenced during crack propagation by the constrained bondline, which changes the value of the critical energy release rate. McCarthy (1999) stated that, for most practical joints, no crack of sufficient size exists. Fracture mechanics of uncracked joints are complex and rely on the concept of inherent flaws, where a flaw size is calibrated with experimental results. This is rarely used in practice and one of the criticisms of this method is that the calculated flaw size can be much larger than the flaw sizes that actually exist.

2.5 NUMERICAL MODELING

Numerical methods provide a general tool for analysis of arbitrary geometries and loading conditions. Among the numerical methods, finite element analysis has been

extensively used with success. This kind of analysis requires the generation of a large set of data in order to obtain reasonably accurate results. However, the cost of large investment in engineering time and computer resources brings more accuracy and visible details compared to other analytical methods, especially for the cases with relatively large stress or strain gradient.

2.5.1 Modeling of Adhesive Joints

Wooley and Carver (1971) made one of the first finite element analyses of a single lap joint. They used plane stress elements, and their results were comparable with those from the Goland and Reissner (1944) solution, the seminal work in the field. Countless studies of various joints have followed this first work. However, quadrilateral elements require a very fine mesh for adequate accuracy. For this reason, efforts have been made to reduce the size of the models. The first approach is made by Yadagiri (1987) who modeled the adhesive layer using only one row of six-node plane elements. Carpenter and Barsoum (1989) modeled the adherends using two-node beam elements, and the adhesive layer by a linear plane element with offset nodes.

The number of degrees of freedom is reduced appreciably by this approach because the adherends and the adhesive use the same nodes. Previously, Amijima and Fujii (1987) developed a similar model but they required the displacements in the adhesive-adherend interfaces to be compatible. Lin and Lin (1993) introduced an element that represents both the adhesive layer and the adherends, and is based on Timoshenko's beam theory. Reddy and Roy (1988) developed a special element for geometric and material nonlinear analyses. Taylor (1996) used a similar approach to model three-dimensional adhesive joints. In this case, the adhesive is modeled using solid 3-D brick elements, and the adherends are represented by plates. Naboulsi and

Mall (1997) modeled the adhesive layer and adherends using shell elements. This approach considers the contribution of the bending stiffness of the adhesive layer. The model, named the three-layer model, gives good results in those cases where the adhesive layer contributes significantly to the bending properties of the joint.

Pandey and Narasimhan (2001) also contributed to the identification of the three-dimensional nature of stresses in adhesively bonded single-lap joints by performing three-dimensional viscoplastic analysis considering material and geometric nonlinearities. Special elements that model the adhesive layer are also developed for three-dimensional analyses. Edlund and Klarbring (1992) developed an element for geometric nonlinear analysis. They assumed that the adhesive layer is of negligible thickness and it has linear elastic behavior. Soh et al. (2002) combined a damage model within numerical analysis and the result agreed well with the experiment. Andruet et al. (2001) developed a model for three-dimensional analysis of adhesive joints based on shell and solid elements. The shell elements are used to model the adherends, and the adhesive layer is modeled as solid elements with offset nodes in the mid-planes of the adherends. The element formulation includes geometric nonlinearities, and thermal and moisture effects.

From the recent research work (Oh et al., 2003; Dean et al., 2004; Du et al., 2004), it can be seen that modeling adhesive joints using two-dimensional elements is still dominant due to its efficiency and accuracy.

2.5.2 Modeling of Bond Failure Process

Chang and Lessard (1991) modeled the composite plate failure under tensional load. They used a user subroutine to change the behavior of the nonlinear material models in a composite laminate. The user subroutine includes the effects of damage

in FE analysis. It is demonstrated that the damage of composite plate under static tensional load could be successfully modeled by comparison with the experimental results. In their analysis, four-node shell elements and eight-node solid elements are respectively employed in the modeling, and it is showed that there was no indistinguishable difference in the results.

Pradhan et al. (1995) analyzed adhesive bonded joints by computing the strain energy release rate for debonding. They considered the interfaces of adherend and adhesive as potential sources for the initiation of bond failure. Paired nodes are generated on the interfaces and released one after another in a proper sequence, modeling gradual crack growth. Failure loads are successfully predicted. Material properties and geometries that might result in extending the load-carrying capacity of these joints are suggested. Similar method is also used in the work of Bogdanovich and Yushanov (1998).

By employing maximum peel and shear strain of the adhesive as failure criteria, Du et al. (2004) simulated the bond failure processes. User subroutine is used to judge whether the failure index reached the critical value in every analysis increment. The multipoint constraints are released when the maximum value of corresponding strain is achieved. In doing so, progressive failure within the adhesive is simulated successfully and the result agreed well with the experimental measurement.

Feih and Shercliffa (2005) predicted adhesive failure of single-L joint through a 2D model in ABAQUS. A zone criterion was found to predict the failure load best for different fillet shapes, while a damage mechanics approach could be applied to simulate the crack growth during adhesive failure.

2.6 CYCLIC BEHAVIOR

2.6.1 Fatigue Behavior of FRP-Steel Joints

Beyond the common strengthening methods, Colombi et al. (2003) carried out research on reinforcing steel members by employing the pre-stressed composite patch. Fatigue tests were performed on 1000×300 mm steel plates with initial cracks. By comparing the fatigue life of different specimens, it was concluded that by applying a tensile stress equal to 632 N/mm² in the CFRP cross-section, the fatigue life was increased by a factor of about five. Even longer fatigue life could be achieved by stiffer CFRP strips with the same pre-stress level.

Jones and Civjan (2003) conducted both experimental and analytical studies to investigate the effectiveness of applying CFRP overlays to steel fatigue tension coupons to prolong fatigue life. Specimens were either notched or with center hole and tested in uniaxial tension. Two sided applications were found very effective, prolonging fatigue life by as much as 115%. Similar application of CFRP materials subsequent to crack propagation extended the remaining fatigue life by approximately 170% without any other means of crack arrest.

2.6.2 Cyclic Performance of FRP Strengthened Steel Beams

Bassetti et al. (2000) tested a cross girder taken from a ninety-one year old bridge after replacing it with a new structure. Tests proved the effectiveness of using prestressed CFRP plates in stopping fatigue cracks originating at rivet holes and preventing further cracking at other locations. First, the cross girder was cracked after 3.5 million cycles using a stress range of 80 N/mm² and a stress ratio of 0.1.

After strengthening the cross girder with five CFRP plates, no crack growth was recorded up to 20 million cycles.

Two full-scale deteriorated S24x80 steel beams that were taken from a decommissioned bridge were tested to 10 million cycles at a stress range of 34 N/mm² in the tension flange of the repaired beam by Miller et al. (2001). The stress range used represented twice the measured stress range at the tension flange of a bridge which was the subject of a future field demonstration project. The beams were strengthened by adhesively bonding 5.3 mm thick, 37 mm wide CFRP plates to cover the top and bottom of the tension flange. The two beams were tested simultaneously using the same test setup. Both beams completed a 10 million cycle fatigue loading course without exhibiting any indication of delamination of the strips or degradation in stiffness of the beams.

Tavakkolizadeh and Saadatmanesh (2003c) investigated the effect of fatigue strength increase of steel girders retrofitted with CFRP patch by testing 21 specimens made of S127x4.5 A36 steel beams. The results showed that the fatigue life of a steel girder could be extended more than three times and the stable crack growth rates could be decreased by 65%.

Moy (2002) investigated the effect of cyclic loading on the curing of epoxy adhesives. Specimens, consisting of 127 x 76UB13 beams were strengthened with 7.6 mm thick high-modulus CFRP strips with a tensile modulus of 310 GPa. The main parameter investigated was the effect of loading the beams during the adhesive curing process to determine the resulting loss of stiffness. Strain distributions across the depth of the section were also recorded, with the result being a linear distribution of strain. Cycling of the load during the curing reduced the flexural stiffness of the beams, although it was found that limiting the maximum adhesive

stress during curing to 1.0 N/mm^2 , resulted in similar behavior to a strengthened beam that had not been loaded during curing.

Two W200x52 steel beams were tested under cyclic loading conditions to evaluate the effectiveness of two different composite strengthening systems for cyclic strengthening of steel bridge by Mosallam (2005). The first strengthening system used commercially available CFRP laminates which were bonded to the central 1.83 m of the beam while the second used a proprietary composite panel which consisted of a hybrid glass/carbon FRP face sheet with fibers oriented in a $0^\circ/90^\circ$ orientation attached to an aluminum honeycomb. The two beams were subjected to 628,000 load cycles and 855,000 load cycles respectively with an average load of 70 kN. Both beams were tested in four-point bending with a span of 3.81 m and a 91 mm constant moment region. The two composite systems increased the ultimate capacity of the beams by 15 percent and 28 percent respectively as compared to an unstrengthened control beam.

Repair of steel members damaged under repeated loadings with FRP composites has been shown to be possible and effective. Although the system is likely to perform satisfactorily under low amplitude fatigue loading, performance of such systems under high amplitude cyclic loading conditions is unknown (Buyukoztuerk et al., 2004). Considering the fact that partial bond failure in such systems is expected under extended cyclic loading and that environmental exposure may adversely affect bond integrity, it is apparent that a thorough investigation of the high-amplitude cyclic load performance of these systems is of critical importance. Currently, no experimental or analytical research on this issue has been reported.

Considering the ever-increasing field applications of this method, there is an urgent need to fill the research gap in this area. Experimental and theoretical research on

cyclic performance of FRP-retrofitted steel members must be performed to ensure the safety of existing structures under repeated loadings and other future applications.

Table 2.1 Comparison of mechanical properties of fibers (Exel, 2004)

Fiber	Unit	Carbon	Glass	Aramid
Density	$[\times 10^3 \text{ kg/m}^3]$	1.5-1.6	1.9-2.0	1.3
Tensile Modulus	$[\text{kN/mm}^2]$	80-400+	38-45	70-75
Tensile strength	$[\text{N/mm}^2]$	1500-3000+	800-1200	800-1500

Figure 2.1 Configuration of common forms of FRP (Tok, 2003)

Figure 2.2 Effect on modulus of the addition of fibers to a matrix (Azom, 2004)

Figure 2.3 Young's Modulus of FRP and other common structural materials
(Buyukoztuerk et al., 2004)

Figure 2.4 Strain-stress behavior of FRP and other common structural materials
(Buyukoztuerk et al., 2004)

(a)

(b)

(c)

(d)

(e)

Figure 2.5 Applications of FRP in strengthening steel beams: (a) Tickford Bridge (Lane and Ward, 2000); (b) Christina Creek Bridge (Miller et al., 2001); (c) Boots Building Nottingham (Garden, 2001); (d) Hythe Bridge (Luke, 2001); (e) Acton Bridge (Moy and Bloodworth, 2007)

(a)

(b)

Figure 2.6 Possible failure types of FRP-strengthened steel members
(Buyukoztuerk et al., 2004)

Figure 2.7 Illustration of point-based failure criteria (Lee and Kong, 2000)

Figure 2.8 Schematic explanation of shearing in adhesive (Hart-Smith, 1973)

Figure 2.9 Deformations and adhesive shear strains in stiffness-unbalanced joints

(Hart-Smith, 1973)

Figure 2.10 Predicted maximum principal stress at failure load (Feih, 2002)

CHAPTER 3

EXPERIMENTAL AND ANALYTICAL STUDY OF FRP-STEEL JOINT

3.1 INTRODUCTION

According to the published research results in the last decade (Adams et al., 1997; Hollaway and Cadei, 2002; Buyukozturk et al., 2004), the dominant failure mode of composite-metal joint is adhesive failure rather than adherend failure. This is the key difference compared to the failure of FRP-retrofitted concrete structures. For such structures, the FRP-concrete bond strength is generally larger than the failure strength of concrete. Therefore, the rip-off failure (surface concrete comes off with FRP) is the most common failure mode for FRP-retrofitted concrete beams. However, for FRP adhesively bonded steel structures, the failure mechanism is totally different. Both steel and FRP materials are much stronger than of the interfacial adhesive bond. Therefore, the adherends in FRP bonded steel structures are unlikely to fail before bond failure. For FRP strengthened or repaired steel structures, due to material discontinuity at the location of bond end or crack tip, the stress concentration in these areas may cause premature adhesive failure before failure of the strengthened structure. On account of this reason, the bond performance of FRP-steel joint needs to be studied carefully.

It is apparent that the bond strength of a composite joint is strongly influenced by its geometry and loading conditions. In the experimental program, bond performance of three different joint types, i.e. double-lap, single-lap and peel joints was investigated. The effect of different parameters on bond strength was analyzed.

Based on the experimental results, numerical analysis was conducted to study stress and strain distributions in FRP-steel joint. The stress distribution in the bondline between steel and FRP was specially analyzed in detail.

3.2 EXPERIMENTAL PROGRAM

To achieve a fundamental understanding of the bond property of FRP strengthened steel structures, experiments on FRP adhesively bonded steel joint with different joint geometries and loading conditions were performed.

3.2.1 Material Properties

The steel material used in the experiment was low carbon steel. Coupon test (ASTM A370, 2006) was performed to get the material properties of the steel. The material properties of the steel are listed in Table 3.1. The “ST” was used to indicate steel coupon and a number was followed to indicate the sequence of tested specimen. For example, “ST-1” indicated the first tested steel coupon. The typical relationship between stress and strain of the steel is illustrated in Figure 3.1. The Instron 4486 universal test machine was used to apply tensile loading. The test setup and failed steel coupon are shown in Figure 3.2. The Young’s modulus of the steel was 196 kN/mm². The yield stress of the steel was 361 N/mm² and the ultimate stress was 516 N/mm².

In this study, a total of five different types of FRP laminates and two types of epoxies were used in the experiments on adhesive FRP-steel joints and FRP strengthened steel beams. The specimens of FRP laminate and solidified epoxy were tested using Instron 5569 universal test machine with correspondence to relevant American standards (ASTM D3039, 2000; ASTM D638, 2003). It is

noteworthy that since each type of laminate has a unique product name which has been used all the time by the manufacturer as well as the customers, for the purpose of convenience and conformity, the product names of these materials were also adopted in this study. The main differences of concern between these materials are the tensile mechanical properties, i.e. tensile strength and tensile modulus. For comparison of the differences between each material, the detailed material properties of these epoxies and FRP laminates are summarized in Table 3.2 and 3.3, respectively. Similar to the name of the steel coupon specimens, each specimen in the test was named with the material name followed by the test sequence. SCH11, SCH41 and UC laminates were CFRP composites. SEH51 and UG were GFRP composites. The epoxy used for the SCH11, SCH41 and SEH51 was a two-component epoxy matrix material TYFO S Saturant. For the other two types of laminates, the epoxy TYFO TC was used. The stress-strain curves of these epoxies and FRP laminates are plotted in Figure 3.3 and 3.4, respectively.

3.2.2 Details of Specimens

With reference to ASTM standards (2000, 2001, 2003) and other relevant research publications (Sheppard et al., 1998; Jiao and Zhao, 2004) the following specimens were designed:

- (a) Double-lap joint
- (b) Single-lap joint
- (c) T-peel joint
- (d) Tubular joint

The configurations of the four types of joint specimens are illustrated in Figures 3.5 to 3.8, respectively. To investigate the bond strength under shear dominant loading

case, double-lap joint specimens were employed as well as tubular joints, because adhesive failure in these joints was believed to be caused mainly by shearing effect in the interface between adhesive and adherend. T-peel and single-lap joint specimens were respectively employed to investigate peel and mixed loading effects. Generally, the thickness of adherends and the fiber orientation have effect on the bond strength of composite joints to some extent. There could be innumerable joint forms if a combination of these two factors was considered. According to the result of Jiao and Zhao (2004), the effect of FRP thickness on bond strength was limited in certain scope. To simplify the problem, only one layer of FRP parallel to the loading orientation was applied to the adherend.

According to different objectives, the specimens were divided into different groups. The first group (group T) was designed to study the effect of surface treatment of the bond performance of FRP-steel joint. The specimen details are shown in Table 3.4. Three different surface treatment methods i.e. brushing, grinding and sandblast were used. Four types of FRP laminates were used in this group. Three specimens were tested for each joint. The letter in the name of each specimen indicated the surface treatment method as follows: A- brushing, B- grinding and C- sandblast. The first number in the specimen name indicated the material type, i.e. SCH11, SCH41, UC and UG indicated by 1 to 4, respectively. The second number is the test sequence of each type of specimen. The comparison of the steel surfaces treated by these methods is shown in Figure 3.9. It is clearly that the steel plate treated by sandblast has the smoothest surface. Only double-lap joint was employed in this group. The geometry and material property of the joint were symmetrical about the central cross section. The thickness of the inner steel adherend was 5mm. The detailed dimensions of the specimen are shown in Figure 3.10.

Specimens in group DS and DU were designed to investigate the bond strengths of double-lap and tubular joints with different bond lengths. To facilitate the comparison between double-lap and tubular joints, the bond width of tubular joint was defined as half of the perimeter of the tube. The specimen details are summarized in Tables 3.5 and 3.6, respectively. In the name of each specimen, the first letter “D” indicated the double-lap joint. The second letter indicated the material type as “S” for SCH41 and “U” for UC. Different bond lengths from 13 mm to 400 mm were indicated by the first number. The test sequence of each type of specimen was indicated by the second number. Similarly, specimens in group S and group P were designed to study the bond strengths of single-lap and T-peel joints with different bond lengths. The specimen details are listed in Tables 3.7 and 3.8, respectively. The specimens are named in same manner as those in Tables 3.5 and 3.6.

3.2.3 Specimen Preparation

For each specimen, the surface of steel plate was mechanically polished first. This process was to make the steel surface free of protrusions or cavities, which may cause voids in the bondline. Before bonding the FRP laminates both steel and FRP surfaces were cleaned with the organic solvent methyl ethyl ketone to remove possible residuals remained, such as rust, oil and dust. Two steel plates were fixed in line in a jig when adhesives and laminates were applied. While the FRP laminates being bonded, they were carefully rolled to ensure that there was no air hump remained. After the FRP laminate was adhered to the steel plate, extra epoxy were cut off carefully and removed from the specimens. The specimens were cured in room temperature for 72 hours before they are tested.

3.2.4 Test Setup and Procedure

For each double-lap and single-lap joint, two strain gauges were placed on the steel and CFRP adherend respectively to monitor the strain increment in the loading process. For the tubular joint, nine strain gauges were placed on the CFRP adherend to investigate the strain variation along the axial direction. The positions of the strain gauges are illustrated in Figure 3.11.

Two types of universal testing machines were used as loading equipment. For double-lap, single-lap and T-peel joints, the Instron 5500 universal electro-mechanical testing system was used. The loading speed for these specimens was 0.01mm/s. For tubular joints, the Instron 5590 universal hydraulic testing system was employed to satisfy the requirement of larger dimensions and higher forces. A loading speed of 0.02mm/s was employed. An experimental set-up consisting of data logger, switch box and computer was used for load, displacement and strain records. Each specimen was loaded monotonically until failure.

3.3 TEST RESULTS AND ANALYSIS

3.3.1 Bond Strength

For same type of joints with different bond area, the ultimate failure loads could be different. Generally, for joints with same bond length, the failure loads are proportional to the bond widths. To make it easier to compare the bond failure resistance capacities of same type of joints with different bond area, a special definition of bond strength in unit bond length is used. For double-lap joint specimens, the bond strength is defined as,

$$S = P/(2b_0) \quad (3.1)$$

where P is the ultimate load for the specimen and b_0 is the bond width. For tubular joints, the bond strength is defined as follows,

$$S = P / (2\pi R) \quad (3.2)$$

where R is the outer radius of steel tube. Similarly, the bond strength in unit length for single-lap and peel joints is defined as,

$$S = P / b_0 \quad (3.3)$$

Based on the experimental results of double-lap and tubular joints, the average bond strengths of different types of joints are listed in Table 3.5 to 3.8, respectively.

3.3.2 Effect of Bond Length

With the experimental results, the trend of bond strength increase was found out. The relationship between bond strength and bond length of joints with SCH41 and UC laminates are shown in Figure 3.12 and 3.13, respectively. It can be seen that the longer the bond length is, the less the bond strength increases. When the bond length is large enough, the strength increase with the rising length tends to be unnoticeable. This phenomenon is quite similar with what exists in the adhesively bonded metal joints (Sheppard et al., 1998).

For single-lap joint, there was a similar trend of bond strength increase. However, for T-peel joint, the change of bond length did not have an obvious effect the bond strength. This is because both the peel and the shear stress are only highly distributed at the end of the bondline. The bond strength of joint with larger bond width tended to be less than that of joint with a smaller one. This is mainly due to the stiffness nonuniformity normal to the laminate surface existed in the FRP laminates and the bond area. The peel strength is sensitive to this normal stiffness. Therefore, the stiffness nonuniformity may lead to instability in the peel strength.

As the bond width increased, the effect of this instability was more pronounced. Consequently, for peel joint specimens, a small bond width is preferred.

For double-lap, tubular and single-lap joints, lap shear strength between FRP laminate and steel plate is an important parameter to evaluate the effectiveness of the adhesive bond. The lap shear strength of bonded joint is defined as the ultimate strength per unit bond area. For double-lap joint, the shear strength is defined as

$$V = P / (2b_0l_0) \quad (3.4)$$

where l_0 is the bond length of the joint. For tubular joint, the shear strength is defined as

$$V = P / (2\pi Rl_0) \quad (3.5)$$

Similarly, the equation of lap shear strength for single-lap joint is:

$$V = P / (b_0l_0) \quad (3.6)$$

By plotting the lap shear strengths of double-lap joints with SCH41 and UC laminates together with their corresponding bond lengths, the relationships between lap shear strength and bond length of joints with SCH41 and UC laminates are illustrated in Figure 3.14 and 3.15, respectively. It is notable that the lap shear strength increasing trend is opposite to that of bond length. The reason for this phenomenon is to be discussed in the last section.

3.3.3 Effect of Surface Treatment

To analyze the effect of the steel surface treatment method on the strength of the FRP-steel double lap joints, the comparison of lap shear strengths of all the specimens bonded with SCH41 and SCH11 laminate systems is illustrated in Figure 3.16 by employing clustered columns. It is clearly shown that the specimens with steel surface treated by brushing had the lowest strength. Grinding and sandblasting

can create much better steel surface for bonding with FRP laminate, while the strengths of the specimens treated by grinding were a little lower than those treated by sandblasting.

Similarly, the comparison of the lap shear strengths of the specimens with UC and UG laminate systems is shown in Figure 3.17. The joints with steel surface treated by wire-brush were still the weakest joints, and those with sandblasted steel surface were strongest. However, for the joints with steel surface treated by grinding, the strength superiority over the specimens with brushed steel surface was not that remarkable. Based on the above comparisons, it can be found that sandblasting is most desirable method to create appropriate steel surface for adhesive bonding with FRP laminates.

3.3.4 Effect of Composite Type

It can be seen that for joints with steel surfaces treated by same polishing method, the maximum loads of the joints with UC and UG laminate systems were much higher than that of the joints with SCH41 and SCH11 systems. This difference was mainly caused by the different material properties of the epoxy materials employed to bond different FRP laminates. For laminates UC and UG, the corresponding epoxy used was TC. The adhesion of this epoxy to steel is better than that of S, which is the accompanying epoxy for SCH41 and SCH11 composite systems.

It is also worth notice the joint with SCH41 laminate had a slightly higher load capacity than that with SCH11 laminate, and the joint with UC laminate was somewhat stronger than the joint with UG laminate. This difference in load capacities was resulted from the stiffness difference of the FRP laminates. FRP-steel double lap joint is an imbalanced joint, i.e. the material properties of the

outer and inner adherends are different. According to the analysis of Hart-Smith (1987), the larger the stiffness difference between the outer and inner adherends, the weaker the bond of the imbalanced joint is. In this study, the stiffness difference between the SCH41 laminate and the steel substrate was smaller than that between the SCH11 laminate and the steel substrate. Therefore, the joint with the SCH41 laminate was stronger compared to the joint with the later when the other properties and conditions are same. For the case of joints bonded with UC and UG laminates, the same mechanism also applies to explain the difference in the load capacities of the joints.

3.3.5 Failure Modes

Failure in the FRP laminate or the steel adherend was not observed in the experiment. All specimens showed an adhesive failure mode. The rupture happened at the adhesive–steel interface. This is because the bond between epoxy and steel surface is the weakest link in the FRP bonded steel structure. The ultimate failure of all specimens always occurred in a sudden manner, without any visible signs or prior warnings. The failure modes of selected specimens are shown in Figure 3.18. The first two photos are the ruptured interfaces between epoxy and steel for specimens with epoxy S and TC, respectively. The epoxy was completely detached from steel surface. This was the failure mode for most specimens. In the last two pictures, the epoxy is not fully ripped off from the steel surface. This is most likely due to the defective bond between FRP and epoxy developed during the bonding process. Such defective bond could have been caused by the insufficiently cleared laminate surface or small air bubbles in-between the laminate surface and the epoxy.

3.3.6 Numerical Analysis

To analyze the adhesively bonded joints tested above, the FE software package ABAQUS (Version 6.5-4) was used in this study. Isoparametric, three-node and four-node elements were employed to create plane strain models of the joints. Material nonlinearity was also incorporated in each analysis to model the stress-strain relationship of the adhesive and the steel adherends. The material stress-strain curves given in Figures 3.1, 3.3 and 3.4 were adopted in the analyses. Sheppard et al. (1997) showed that the Von Mises yield criterion was suitable to model the stress-strain behavior of the adhesive used in these joints. Consequently in this work, the criterion was also used for the adhesive layer and for the steel adherends.

Both the adherends and the adhesive were modeled as separate parts and linked together in the analysis. A pair of nodes at each identical location existed at the material interfaces, one belonging to the adherend, the other to the adhesive. All degrees of freedom of the corresponding nodes were constrained to identical displacements. This approach was necessary to obtain accurate stress and failure ratio results at the interface, i.e. to obtain stress jumps at the interface due to the stiffness jumps between the different materials.

In the analysis, it was important to take the geometric nonlinearity into account. The relative displacement of FRP and steel adherends changed the stress distributions in adherends and adhesive under loading. Hence, the stress distributions at small loads often differed considerably from the distributions at high loads. Additionally, it is also important to take the material nonlinearities of the adherends and adhesives into account. Although most parts of steel adherends remained elastic in the whole analysis process, it should not be neglected that in adherend near the joint end, a

small plastic area may exist due to high stress concentration. This effect of the small plastic area on stress distribution in the adhesive layer was not negligible.

In the FE analysis of FPR-steel joints, the issue of most concern was the stress and strain distribution in the adhesive layer, because the failure mode of the joints was adhesive failure. The adhesive layer was very thin compared with the thickness of the adherends but, to achieve reliable results, it was necessary to use fine mesh for the adhesive and the regions in the adherends close to the adhesive. Other further regions, such as the adherends outside the joint overlap, can be modeled with a coarser mesh. On the other hand, too fine mesh may lead to rather high number of degrees of freedom, which is time-consuming and it further complicates the pre- and post-processing of the analysis. With reference to Sheppard et al. (1998), the adhesive layer should be meshed in the size of adhesive thickness, and the relevant FE analysis results could be quite good. Accordingly, one layer of elements was used for the adhesive; and four to seven layers of elements were employed for the adherends.

To verify the validity of the numerical model, the strain distributions along the length of FRP laminate from numerical and experimental results are compared in Figure 3.19. L is the distance from the end of the laminate. It can be seen in the major part of the FRP laminate the strain distributed evenly, while at the FRP and steel ends the strain varied sharply, especially at the later one. Because the cross-section magnitude of stiffness of the internal steel tube was larger than that of the FRP laminate, more loads were shared by steel than FRP in the middle region where FRP and steel bear together. At the end of the steel tube, load transferred into FRP laminate was much more than that at the FRP end, so the stress/strain in the FRP laminate increases even faster.

The relationships between load and displacement of double-lap, single-lap and peel joints are illustrated in Figures 3.20 to 3.22, respectively. The specimens with 50 mm bond length are employed for the comparison between numerical and experimental results. It can be seen that the relationship between load and deflection of the numerical analysis agree well with that of the experimental result. The difference between numerical and experimental curves tends to be enlarged when the deflection increases. This difference is mainly caused by the difference between the constitutive relationships of material used in the numerical analysis and that of the real material. With the above comparisons, it is deemed that the numerical model is valid to be used in the subsequent analysis for joints with bond length of 50 mm and maximum deformation is no more than 0.25 mm.

3.3.7 Stress/Strain Distribution

In FRP-retrofitted structures, the most significant role the adhesive plays is force transfer. Generally, when the original structure is retrofitted by FRP, the external loads do not exert on the laminates directly. The stress in the longitudinal direction in FRP comes from the original structure by the transfer of adhesive layer, which undergoes large shear deformation. Therefore, the shear stress and strain distributions in adhesive should be given much attention. Hart-Smith (1980) discussed that only for very short overlap test coupons is the strength a function of the bond area. For long overlap bonded joints the strength is not uniform, with the great majority of the load carried by the plastic adhesive zones separated by the lightly loaded elastic zone. The width of these plastic zones is constant regardless of the overlap. However, if one of the adherends had been less stiff than the other, the load transfer would not have been balanced and the adhesive could be critically strained at one end (from which the softer adherend extended) before a significant load transfer had developed at the other end.

Based on the FE analysis of the tubular joint with 200 mm bond length, the shear stress in the adhesive layer and the tension stress in the midline of steel and FRP are plotted in Figure 3.23. It can be seen that from the left to the right, the stress in steel dropped from maximum value to zero. To keep the section load balanced, the stress in FRP increased so that the stress transfers gradually from steel to FRP. The major part of load transferred in the area near the joint ends, and in the middle section of the joint the stresses were more evenly distributed according to the stiffness of steel and FRP. Therefore, the value of shear stress in the adhesive end was very large as a result of stress transfer between steel and FRP. In the middle section where the stresses in steel and FRP were more evenly distributed, the shear stress in adhesive was close to zero.

Based on the FE analysis of double-lap and tubular joints with different bond lengths, the shear stress distributions in the adhesive layers are shown in Figure 3.24. It can be seen that for the joint with relatively short bond length, the maximum stress in the joint end did not reach the maximum shear stress of the adhesive at the moment the joint fails, and the stress in the middle section of the joint was of high magnitude. With an increasing bond length, the stress at the joint end went up. For the joint with 200 mm bond length, the maximum shear stress had been attained in the joint end before the joint failed. The stress in the middle section kept relatively low value till the joint failed. This was similar to the stress distribution of conventional symmetric double-lap metal joints. Compared to the conventional symmetric double-lap metal joints, an essential difference of the FRP-steel joint is that the stress magnitudes are not equal in each end of the joint. It is shown that the stress magnitude at the left end of the joint was two more times less than that at the right. This phenomenon can be explained by the stiffness difference between the upper and the lower adherends. Since FRP laminate is less stiff than steel, the stress

at the joint end where the steel adherend terminates is much higher than the stress at the other end. Therefore, the bond failure initiated from the end where the steel adherend terminates. This conclusion agrees with the test observation.

In the adhesive layer, the normal stress and strain at the joint end are not negligible. Generally the magnitude of peel stress and strain is much less than that of relevant shear components for in-plane loaded joints. However, for adhesive materials, the volume deformation capacity is not as good as shape deformation capacity, which means the magnitude of normal strain to cause the adhesive failure could be lower than that of the shear strain. Further analysis and discussion on this issue are presented in the next section.

Generally, the failure of adhesive joints always initiates from the joint end no matter what the joint geometry is. Based on the FE analysis of different joints, the typical shear and normal strain distributions in the double-lap, single-lap and T-peel joints are illustrated in Figures 3.25 to 3.27. It is clear that both shear and normal strain in the adhesive had prominent magnitude at the joint end. Furthermore, it can be seen that the adhesive failure of all the joints was the result of the combining effect of shear and normal stresses.

The variations of both shear forces and moments along the bondline together with the distributions of shear and normal stresses along the bondline of double-lap, single-lap and T-peel joints are illustrated in Figures 3.28 to 3.33. It can be seen that the shear stress at the end of the bondline of double-lap joint was much higher than the normal stress. On the contrary, the shear stress was lower than the normal stress for T-peel joint. It is obvious that the difference of distributions of different stress components was caused by the different loading conditions. For double-joint, the direction of external force was parallel to that of the bondline, so the force

transferred from steel plate to FRP laminate was mainly realized by the shear deformation of the adhesive. For similar reason, the large adhesive deformation normal to the bondline results in a higher positive normal stress. For single-lap joint, the resultant forces in FRP laminate and steel plate were not in line and thus an internal moment was produced. This internal moment caused bending of both FRP and steel adherends. As a result, the normal stress at the end of the single-lap joint was much higher than that of double-lap joint. From the above analysis, it can be seen that the stress distributions in double-lap, single-lap and peel joints were quite different. Three different combinations of shear and normal stress components, i.e. high shear with low normal stress, high shear with high normal stress and low shear with high normal stress were exhibited in these joints respectively. This feature existed in different types of joints can be utilized to find out the correlative relationship between different stress, strain or energy components in FRP strengthened steel structures. The details of further study on this issue are discussed in Chapter 6.

Table 3.1 Material properties of steel

Coupon	f_y (N/mm ²)				E (kN/mm ²)			
	Test	Ave	STD	CV(%)	Test	Ave	STD	CV(%)
ST-1	362.5	360.8	8.5	2.4	201.9	196.1	4.3	2.2
ST-2	353.2				197.2			
ST-3	348.5				198.2			
ST-4	369.3				196.6			
ST-5	369.8				188.7			

Note: Letter in coupon name refers to material (ST- Steel) and number refers to test sequence; f_y - Yield strength of steel; E - Young's modulus of steel; Ave- Average; STD- Standard deviation; CV- Coefficient of variation.

Table 3.2 Material properties of epoxy

Coupon	f_a (N/mm ²)				E_a (kN/mm ²)			
	Test	Ave	STD	CV(%)	Test	Ave	STD	CV(%)
S-1	39.4	45.8	4.3	9.4	2.16	2.17	0.10	4.8
S-2	49.4				2.01			
S-3	43.8				2.12			
S-4	45.0				2.28			
S-5	51.6				2.30			
TC-1	16.9	15.5	1.2	8.0	0.64	0.63	0.05	7.2
TC-2	14.4				0.57			
TC-3	15.5				0.71			
TC-4	13.9				0.63			
TC-5	16.9				0.61			

Note: Letter in coupon name refers to material and number refers to test sequence; f_a - Yield strength of epoxy; E_a - Young's modulus of epoxy; Ave- Average; STD- Standard deviation; CV- Coefficient of variation.

Table 3.3 Material properties of FRP laminate

Coupon	f_f (N/mm ²)				E_f (kN/mm ²)			
	Test	Ave	STD	CV (%)	Test	Ave	STD	CV (%)
SCH11-1	1221	1196	55	4.6	111.0	107.7	3.4	3.2
SCH11-2	1244				112.6			
SCH11-3	1168				104.9			
SCH11-4	1101				105.8			
SCH11-5	1248				104.1			
SCH41-1	922	911	31	3.4	82.3	83.0	3.0	3.6
SCH41-2	954				85.5			
SCH41-3	886				83.6			
SCH41-4	866				85.9			
SCH41-5	926				77.5			
UC-1	2478	2492	108	4.3	150.6	150.8	3.2	2.3
UC-2	2644				147.8			
UC-3	2307				155.2			
UC-4	2523				146.4			
UC-5	2506				154.1			
SEH51-1	631	616	19	3.1	31.1	31.8	1.5	4.7
SEH51-2	640				32.2			
SEH51-3	597				29.3			
SEH51-4	619				32.9			
SEH51-5	591				33.5			
UG-1	912	884	19	2.2	45.3	43.7	1.3	3.0
UG-2	897				41.7			
UG-3	874				42.8			
UG-4	856				44.2			
UG-5	880				44.6			

Note: Letter in coupon name refers to material and number refers to test sequence; f_f - Yield strength of FRP; E_f - Young's modulus of FRP; Ave- Average; STD- Standard deviation; CV- Coefficient of variation.

Table 3.4 Specimen details and experimental results (group T)

Specimen	FRP	Treatment	P (kN)	V (N/mm ²)	V_{Ave} (N/mm ²)
A11	SCH11	Brushing	7.5	0.4	0.5
A12	SCH11	Brushing	9.2	0.5	
A13	SCH11	Brushing	10.8	0.5	
A21	SCH41	Brushing	10.3	0.5	0.6
A22	SCH41	Brushing	10.6	0.5	
A23	SCH41	Brushing	12.9	0.6	
A31	UC	Brushing	41.1	2.0	2.3
A32	UC	Brushing	47.5	2.3	
A33	UC	Brushing	53.9	2.7	
A41	UG	Brushing	37.8	1.9	2.2
A42	UG	Brushing	46.3	2.3	
A43	UG	Brushing	48.7	2.4	
B11	SCH11	Grinding	26.7	1.3	1.5
B12	SCH11	Grinding	30.5	1.5	
B13	SCH11	Grinding	31.7	1.6	
B21	SCH41	Grinding	29.7	1.5	1.8
B22	SCH41	Grinding	36.2	1.8	
B23	SCH41	Grinding	41.1	2.0	
B31	UC	Grinding	72.4	3.6	4.0
B32	UC	Grinding	83.5	4.1	
B33	UC	Grinding	88.2	4.3	
B41	UG	Grinding	79.1	3.2	3.8
B42	UG	Grinding	88.5	3.9	
B43	UG	Grinding	88.2	4.4	
C11	SCH11	Sandblast	31.8	1.6	1.7
C12	SCH11	Sandblast	34.8	1.7	
C13	SCH11	Sandblast	37.2	1.8	
C21	SCH41	Sandblast	36.4	1.8	2.0
C22	SCH41	Sandblast	41.4	2.0	
C23	SCH41	Sandblast	43.7	2.1	
C31	UC	Sandblast	103.6	5.1	5.8
C32	UC	Sandblast	112.4	5.5	
C33	UC	Sandblast	138.7	6.8	
C41	UG	Sandblast	99.4	4.9	5.5
C42	UG	Sandblast	117.6	5.8	
C43	UG	Sandblast	119.1	5.9	

Note: Letter in coupon name refers to surface treatment type (A- Brushing, B- Grinding, C- Sandblast) and two numbers refer to material (1- SCH11, 2- SCH41, 3- UC, 4- UG) and test sequence respectively; P - Ultimate load; V - Shear strength; V_{Ave} - Average shear strength.

Table 3.5 Specimen details and experimental results (group DS)

Specimen	l_0 (mm)	b_0 (mm)	P (kN)	S (kN/m)	V (N/mm ²)	V_{Ave} (N/mm ²)
DS11	13	25	4.5	90	6.9	6.1
DS12	13	25	3.5	69	5.3	
DS13	13	25	3.9	78	6.0	
DS21	50	50	21.2	212	4.2	4.0
DS22	50	50	22.5	225	4.5	
DS23	50	50	19.5	195	3.9	
DS24	50	50	16.0	160	3.2	
DS31	75	50	27.4	274	3.7	3.5
DS32	75	50	24.8	248	3.3	
DS33	75	50	27.9	279	3.7	
DS34	75	50	25.4	254	3.4	
DS35	75	50	25.5	255	3.4	
DS41	100	50	31.8	318	3.2	3.4
DS42	100	50	37.1	371	3.7	
DS43	100	50	35.2	352	3.5	
DS44	100	50	32.5	325	3.3	
DS51	150	50	35.2	352	2.3	2.4
DS52	150	50	37.2	372	2.5	
DS53	150	50	30.7	307	2.0	
DS54	150	50	40.8	408	2.7	
DS61	200	50	40.5	405	2.0	2.1
DS62	200	50	46.7	467	2.3	
DS63	200	50	39.8	398	2.0	
DS64	200	50	41.9	419	2.1	
DS65	200	94	83.8	449	2.2	
DS66	200	94	72.2	383	1.9	
DS71	250	50	52.9	529	2.1	1.9
DS72	250	50	46.5	465	1.9	
DS73	250	50	47.5	475	1.9	
DS74	250	50	41.5	415	1.7	
DS81	400	94	87.6	465	1.2	1.2
DS82	400	94	97.6	521	1.3	

Note: Letter in coupon name refers to group name (DS) and two numbers refer to bond length and test sequence respectively; l_0 - Bond length; b_0 - Bond width; P - Ultimate load; S - Bond strength; V - Shear strength; V_{Ave} - Average shear strength.

Table 3.6 Specimen details and experimental results (group DU)

Specimen	l_0 (mm)	b_0 (mm)	P (kN)	S (kN/m)	V (N/mm ²)	V_{Ave} (N/mm ²)
DU11	25	25	15.8	315	12.6	12.7
DU12	25	25	14.6	291	11.6	
DU13	25	25	15.3	305	12.2	
DU14	25	25	17.7	354	14.2	
DU21	50	50	51.6	516	10.3	9.1
DU22	50	50	47.2	472	9.4	
DU23	50	50	43.2	432	8.6	
DU24	50	50	41.0	410	8.2	
DU31	75	50	53.9	539	7.2	7.4
DU32	75	50	60.4	604	8.1	
DU33	75	50	54.5	545	7.3	
DU34	75	50	55.4	554	7.4	
DU35	75	50	52.0	520	6.9	
DU41	100	50	66.3	663	6.6	6.4
DU42	100	50	70.9	709	7.1	
DU43	100	50	53.0	530	5.3	
DU44	100	50	64.1	641	6.4	
DU51	150	50	59.6	596	4.0	5.0
DU52	150	50	71.2	712	4.7	
DU53	150	50	85.0	850	5.7	
DU54	150	50	81.2	812	5.4	
DU61	200	50	71.0	710	3.6	3.9
DU62	200	50	73.4	734	3.7	
DU63	200	50	80.6	806	4.0	
DU64	200	50	89.7	897	4.5	
DU71	250	50	70.5	705	2.8	3.2
DU72	250	50	88.6	886	3.5	
DU73	250	50	82.2	822	3.3	
DU74	250	50	82.8	828	3.3	

Note: Letter in coupon name refers to group name (DU) and two numbers refer to bond length and test sequence respectively; l_0 - Bond length; b_0 - Bond width; P - Ultimate load; S - Bond strength; V - Shear strength; V_{Ave} - Average shear strength.

Table 3.7 Specimen details and experimental results (group S)

Specimen	l_0 (mm)	b_0 (mm)	P (kN)	S (kN/m)	V (N/mm ²)	V_{Ave} (N/mm ²)
SS11	25	25	2.5	100.0	4.0	5.0
SS12	25	25	3.0	120.0	4.8	
SS13	25	25	3.2	128.0	5.1	
SS14	25	25	3.9	156.0	6.2	
SS21	50	50	8.4	168.0	3.4	3.8
SS22	50	50	9.2	184.0	3.7	
SS23	50	50	9.7	194.0	3.9	
SS24	50	50	10.2	204.0	4.1	
SS31	200	50	14.5	290.0	1.5	1.6
SS32	200	50	16.0	320.0	1.6	
SS33	200	50	16.5	330.0	1.7	
SS34	200	50	17.2	344.0	1.7	
SU11	25	25	4.6	184.0	7.4	9.3
SU12	25	25	5.7	228.0	9.1	
SU13	25	25	5.9	236.0	9.4	
SU14	25	25	7.1	284.0	11.4	
SU21	50	50	15.8	316.0	6.3	7.5
SU22	50	50	18.5	370.0	7.4	
SU23	50	50	19.1	382.0	7.6	
SU24	50	50	22.0	440.0	8.8	
SU31	200	50	25.8	516.0	2.6	3.0
SU32	200	50	28.8	576.0	2.9	
SU33	200	50	29.2	584.0	2.9	
SU34	200	50	34.3	686.0	3.4	

Note: In each coupon name, two letters refer to group name and material (S- SCH41, U- UC) respectively; two numbers refer to bond length and test sequence respectively; l_0 - Bond length; b_0 - Bond width; P - Ultimate load; S - Bond strength; V - Shear strength; V_{Ave} - Average shear strength.

Table 3.8 Specimen details and experimental results (group P)

Specimen	l_0 (mm)	b_0 (mm)	FRP	P (kN)	S (kN/m)	S_{Ave} (kN/m)
PS11	50	25	SCH41	0.18	7.2	9.7
PS12	50	25	SCH41	0.24	9.6	
PS13	50	25	SCH41	0.26	10.4	
PS14	50	25	SCH41	0.29	11.6	
PS21	100	50	SCH41	0.38	7.6	8.6
PS22	100	50	SCH41	0.40	8.0	
PS23	100	50	SCH41	0.43	8.6	
PS24	100	50	SCH41	0.51	10.2	
PU11	50	25	UC	0.43	17.2	19.9
PU12	50	25	UC	0.44	17.6	
PU13	50	25	UC	0.53	21.2	
PU14	50	25	UC	0.59	23.6	
PU21	100	50	UC	0.74	14.8	18.9
PU22	100	50	UC	0.94	18.8	
PU23	100	50	UC	1.01	20.2	
PU24	100	50	UC	1.09	21.8	

Note: In each coupon name, two letters refer to group name and material (S- SCH41, U- UC) respectively; numbers refer to bond length and test sequence respectively; l_0 - Bond length; b_0 - Bond width; P - Ultimate load; S - Bond strength; S_{Ave} - Average bond strength.

Figure 3.1 Typical stress-strain curve from steel coupon test

(a) Steel coupon

(b) FRP coupon

(c) Epoxy coupon

(d) Instron 5569 universal test machine

(e) Instron 4486 universal test machine

Figure 3.2 Material test setup

Figure 3.3 Stress-strain curves of epoxies

Figure 3.4 Stress-strain curves of FRP laminates

Figure 3.5 Double-lap joint ($b_0 = 25\sim 50\text{mm}$, $l_0 = 13\sim 250\text{mm}$, $t = 2\sim 5\text{mm}$)

Figure 3.6 Single-lap joint ($b_0 = 25\sim 50\text{mm}$, $l_0 = 25\sim 200\text{mm}$, $t = 2\sim 5\text{mm}$)

Figure 3.7 T-peel joint ($b_0 = 25\sim 50\text{mm}$, $l_0 = 50\sim 100\text{mm}$, $t = 5\text{mm}$)

Figure 3.8 Tubular joint ($l_0 = 200\sim 400\text{mm}$)

Figure 3.9 Steel surfaces treated with (a) wire-brush (b) grind (c) sandblast

Figure 3.10 Dimensions of specimens tested in group T (unit: mm)

Figure 3.11 Illustration of the positions of strain gauges on FRP laminate (unit: mm)

Figure 3.12 Relationship between bond strength and bond length

Figure 3.13 Relationship between bond strength and bond length

Figure 3.14 Relationship between shear strength and bond length

Figure 3.15 Relationship between shear strength and bond length

Figure 3.16 Comparison of lap shear strengths of joints with SCH41 and SCH11 laminates

Figure 3.17 Comparison of lap shear strengths of joints with UC and UG laminates

Figure 3.18 Failure modes of joints (a), (b) with no epoxy detached from FRP
(c), (d) with partial epoxy detached from FRP

Figure 3.19 Strain distribution along the FRP laminate

Figure 3.20 Load versus displacement for double-lap joint

Figure 3.21 Load versus displacement for single-lap joint

Figure 3.22 Load versus displacement for peel joint

Figure 3.23 Stress distributions in adherends and adhesive

Figure 3.24 Shear stress distribution in double-lap joints

(a) Normal strain

(b) Shear strain

Figure 3.25 Normal and shear strain in double-lap joint end

(a) Normal strain

(b) Shear strain

Figure 3.26 Normal and shear strain in single-lap joint end

(a) Normal strain

(b) Shear strain

Figure 3.27 Normal strain and shear strain in peel joint end

Figure 3.28 Variation of moment and shear force along bondline of double-lap joint

Figure 3.29 Stress distribution along bondline of double-lap joint

Figure 3.30 Variation of moment and shear force along bondline of single-lap joint

Figure 3.31 Stress distribution along bondline of single-lap joint

Figure 3.32 Variation of moment and shear force along bondline of T-peel joint

Figure 3.33 Stress distribution along bondline of T-peel joint

CHAPTER 4

EXPERIMENTAL STUDY OF FRP STRENGTHENED STEEL BEAMS UNDER STATIC LOADING

4.1 INTRODUCTION

The traditional methods for strengthening steel structures are usually to weld or to mechanically fasten additional steel plates to existing systems. Although they have been used for an age, these techniques have some inevitable disadvantages, which include the difficulty in installation of heavy steel members, increased self weight, weld-induced residual stress, the potential for weld fatigue cracking at the cover plate ends and vulnerability to corrosion. On the other hand, the superior mechanical, fatigue and life-time properties of FRP composites make them ideal candidates for the repairing and strengthening of steel bridge, building and other metallic structures in civil engineering. However, research and development on relevant techniques are still quite limited up to date, not to mention extensive applications in practical structural engineering.

The objective of current experimental program is to investigate the effectiveness of external bonding of FRP laminates on enhancing flexural capacity of steel beams under static loading. Experiments on both strengthening of intact beams and repair of damaged beams were conducted. A total of 36 full-scale specimens have been tested under static loading. According to different objectives, three groups of specimens were designed. In group A, intact steel beams were strengthened with FRP laminates. The impact of different strengthening parameters on the enhanced flexural capacity of the beams was the main concern of this group. Artificially

damaged steel beams were used to fabricate the specimens in group B, because the effectiveness of repair and retrofit of degraded steel beams was intended to be checked out. Due to the importance of the bond performance in FRP adhesively strengthened steel structures as mentioned earlier, the specimens in group C were designed to investigate the relationship between bond failure load and different material and geometrical parameters.

4.2 EXPERIMENTAL PROGRAM

4.1.1 Material Properties

The steel beams used in the experiment were wide-flange low-carbon beams. Coupon tests (ASTM 2006) were performed to get the material property of the steel beams. Beams with three types of cross sections were used in the experiments. The schematic of the cross section dimension of the steel beams is illustrated in Figure 4.1. For the first seven beams in group A, the depth of the beam was $d = 177.8$ mm and the depth between fillets was $h = 146.8$ mm. The width of the beam flange was $b_f = 101.2$ mm. The thicknesses of the flange and the web were $t_f = 7.9$ mm and $t_w = 4.8$ mm, respectively. The Young's modulus was 186 kN/mm^2 , the yield strength was 320 N/mm^2 and the ultimate strength was 491 N/mm^2 . For the seven beams in group B and the last four beams in group A, the depth of the beam was $d = 302.5$ mm and the distance between fillets was $h = 275.8$ mm. The width of the beam flange was $b_f = 100.8$ mm. The thicknesses of the flange and the web were $t_f = 5.7$ mm and $t_w = 5.1$ mm, respectively. The Young's modulus was 206 kN/mm^2 and the yield and the ultimate strengths were 376 N/mm^2 and 505 N/mm^2 , respectively. For the eighteen specimens in group C, the depth of the beams was $d = 175$ mm and the distance between fillets was $h = 143$ mm. The width of the beam flange was $b_f = 90$ mm. The thicknesses of the flange and the web were $t_f = 8$ mm and $t_w = 5$ mm,

respectively. The Young's modulus was 192 kN/mm^2 and the yield and the ultimate strengths were 334 N/mm^2 and 483 N/mm^2 , respectively. For FRP laminates and epoxy adhesives, the material properties were same as those used in the experiment of FRP-steel joints.

4.1.2 Specimen Details

In Figure 4.2, the front view of typical specimen is illustrated. Total length of the steel beam and the bond length of laminate are l and l_0 , respectively. The distance between loading points is l_l . On each side, the length from support to loading point is l_2 . The CFRP strip is bonded to the bottom flange of the steel beam. The width of the CFRP strip is the same as that of the flange. The strengthened beam is symmetric in all the dimensions.

According to the different objectives, the specimens were divided into three groups. Specimens in group A were designed to investigate the strengthening effects of FRP for intact steel beams. The details of specimens are listed in Table 4.1. The first seven specimens were fabricated from steel beams with the first section size to investigate the strengthening effects of different types of FRP laminates. The other specimens were designed to study the strengthening effect of different layers of laminates. Strain variation along the depth of the web was measured during the experiment. It is clear that the relative precision of strain measurement can be improved by increasing the depth of the web since the strain gauge is to be installed manually. To have a relative large depth, these specimens were made from steel beams with the second section size. An additional steel plate was welded to the top flange of each specimen to prevent it failing first since up to six layers of FRP laminates were used. Specimens in group B are designed to explore the strengthening effect of FRP laminates on damaged steel beams. The specimen

details are shown in Table 4.2. To simulate damage and loss of load carrying capacity of steel beam, a 20 mm wide notch was created on bottom flange at mid-span of each specimen. Bond failure was the major concern when specimens in group C were designed. The details of specimens in group C are tabulated in Table 4.3. Specimens were designed to study the effect of different strengthening parameters on the bond failure load of the FRP strengthened steel beams. These parameters include laminate thickness, epoxy thickness, geometry of bond end and bond length. For most specimens, the length of steel beam was 1800 mm. For the last five specimens, the effect of bond length on bond failure load is to be investigated. To have a relative large difference for specimens with various bond lengths, beams with 2400 mm length were employed.

4.1.3 Preparation

To prevent local failure of specimen under point load, all the steel beams were reinforced with stiffeners. Four steel plates were welded to each side of the web where the supports and loading points were located. To investigate the strengthening effect of CFRP laminates on damaged members, the steel beams used to fabricate specimens in group B were artificially damaged. The damage of steel beam was produced by removing a small portion of tension flange from the beam. At mid-span of the steel beam, a 20 mm wide notch on tension flange was created to simulate the serious loss of bottom section. For the last four specimens in group A, 10 mm thick steel plate was welded to the top of the beam. The plate had same size as the flange in the other two dimensions. The purpose of welding this additional plate was to prevent the top flange from failing first under bending. Because for these specimens the bonded FRP laminates are up to six layers, which are more than that of the other specimens, local buckling in the top flange could

probably happen under compression before the bottom flange yielding. These two configurations of steel beams are illustrated in Figure 4.3.

After machining of the steel beams was finished, FRP laminates were tailored to proper sizes for external bonding. To guarantee the quality of the bond between steel and FRP laminate, the bottom surface of steel beams was manually ground. Both the surfaces of FRP laminate and steel beam were cleaned with methyl ethyl ketone to remove rust, oil and dust. Then the epoxy was uniformly applied to the bottom of steel beam as well as FRP laminate surface. The next step was to fix FRP laminates to the steel beam followed by pressing the laminates to even the contact and to extrude possible air bubble between the steel beam and the laminates. After removing the excessive epoxy, the specimens were left untouched at room temperature for 72 hours for curing. For specimens A6 and B5, a pair of mechanical clamps was installed at both ends of the laminate before solidification of the epoxy. A plan view of the clamp assembly is shown in Figure 4.4. The procedure of bonding FRP laminates to steel beams was composed of several steps as shown in Figure 4.5.

4.1.4 Test Setup

All specimens were tested under four-point bending. The schematic and picture of experimental setup are shown in Figure 4.6 and 4.7, respectively. The compression force came from the hydraulic jack with 1000 kN capacity. The jack was fixed to the overhead rigid frame. Transformation from one loading point to two was realized by use of a spreader beam and two rockers between the specimen and the spreader beam. The specimen was supported by a rocker bearing and a pin bearing at the beam ends. Two solid blocks made of high strength concrete were used as underlay to provide reaction to the loading system. To measure vertical

displacements of the specimen, three linear variable displacement transducers were mounted to the specimen bottom. One was positioned at the mid-span and two at the points below the inner stiffeners.

4.1.5 Instrumentation

To measure the strains distributed in steel beam as well as FRP laminate, electrical-resistance strain gauges were mounted to different locations on each specimen. For the first seven specimens in group A, twenty-six strain gauges were used. The locations of strain gauges for a typical specimen are shown in Figure 4.8 (a). Strain gauges 2, 6, 10, 14 are not visible because they are on the other side of the steel web and exactly opposite to gauges 1, 5, 9, 13, respectively. For the other specimens in group A, thirteen strain gauges were used as shown in Figure 4.8 (b). For specimens in group B, the strain gauge on inner surface of tension flange was not available due to the existence of notch. The strain gauges on the web of the steel beam were evenly distributed. The purpose of this strain gauge placement was to collect enough strain data for the subsequent intensive analysis of stress distribution and neutral axial variation under loading. A linear variable displacement transducer LVDT was positioned on the web center at the mid-span of the beam to measure the horizontal displacement under loading. Three LVDTs were positioned at the beam bottom to measure vertical deflections at mid-span and two quarter points. The compression load was measured using load cell. All the data of strain, displacement and load were recorded with a data logger interfacing with a computer through special software.

All specimens were tested under monotonic loading. The test data collected from load cell, LVDT and strain gauge were transferred to the data logger and automatically stored in a computer. In the elastic region, data were collected at

specified load levels. After yielding, they were collected at specified deflection levels.

4.3 EXPERIMENTAL RESULTS

Based on the experimental data and observation, the static behavior of the FRP strengthened steel beams was analyzed. Effects of different parameters on the load carrying capacities of the strengthened beams were compared. With relevant processes observed in the experiments, different failure modes were illustrated. The mechanism of enhancement of flexural capacity by bonding of FRP laminates was extensively discussed based on the analysis of strain and stress distribution and variation in the specimens.

4.3.1 Load Carrying Capacity of FRP Strengthened Undamaged Beams

The load-carrying capacity of each specimen in group A is listed in Table 4.4. It can be seen that with different types of one layer FRP laminates externally bonded, the steel beams got an increase of maximum load from five to eighteen percent. For specimens with additional plate on top flange, the increase was from two to nine percent. The relationships between load and deflection of the beams bonded with wet lay-up laminates are illustrated in Figure 4.9. The beam strengthened with SCH41 laminate had the maximum load increase.

In Figure 4.10, the curves show the relationships between the load and the mid-span deflection of beams strengthened with precured FRP laminates. It can be seen that the beam with UC laminate had the maximum ultimate load increase of nineteen percent. The laminate UG was not as effective as UC in enhancing ultimate load-carrying capacity.

The deflections of specimens with different fiber types under the same load level of 49 kNm, which is the yield strength of the steel beam, are compared in Figure 4.11. The specimen without reinforcement had the maximum mid-span deflection of 9.3 mm. The deflection was reduced to 8.4 mm for the beam strengthened with precured glass fiber reinforced laminate. When precured carbon fiber laminate was bonded to the steel beam, the mid-span deflection was only 7.2 mm, which was the lowest.

The relationships between the load and the mid-span deflection of specimens with different layers of laminates are illustrated in Figure 4.12. It is found that the load increase in the elastic region was not remarkable for the strengthened beams with additional plate welded on top. When the beams underwent plastic deformation gradually, the effect of load enhancement became more noticeable. For the last specimen, there was an obvious drop of load in the plastic regions due to local debonding. This unexpected debonding is believed to be caused by the bonding deficiency. Since the six layers of laminates were manually bonded to the steel beam, the cumulative deficiency caused by human factors was more serious. The comparison of load increases of beams with different layers of laminates is illustrated in Figure 4.13. The load at $D = L/360$ was within elastic region. At the other two deflection levels, the loads fell into plastic region. It is clearly shown that the load carrying capacity of the strengthened beams had been improved much more in plastic region than in elastic region. At the first two deflection levels, the loads of strengthened beams tended to increase linearly with the increase of number of laminate layers. At $D = L/120$, the curve is far from straight because local bond failure happened for the specimen with six layers of laminates.

From the above analyses it can be seen that the maximum increase of load carrying capacity of the steel beam was obtained by external bonding with CFRP laminates.

For all the specimens with external bonded FRP laminates, the load carrying capacities got more enhancements in plastic deformation region than in elastic region. This is because FRP does not yield like steel so it continued to bear the force after the steel yield.

4.3.2 Load Carrying Capacity of FRP Repaired Damaged Beams

The experimental results of specimens with damaged steel sections are listed in Table 4.5. Strengthened with two layers of UC laminates, specimen B7 had the largest load increase of sixty-one percent. The load increase at $L/D = 360$ compared to the un-strengthened beam was forty-nine percent. The relationships between the load and the mid-span deflection of specimens repaired with wet lay-up laminates are shown in Figure 4.14. For the control beam, the transition from elastic to plastic region was so smooth that there was almost no distinct yield region. For the second specimen, the curve has a similar shape with the first while the load capacity was a little higher. For the specimen with SCH41 laminate, there was a distinct elastic region together with an abrupt load drop. For the first specimen, the web bottom began to undergo plastic deformation at a very low load level and deformed rapidly due to the lack of a portion of tension flange. For the specimens with external bonding of FRP laminates, there was great stress concentration in the adhesive layer at the mid-span due to the existence of the notch. Because the bond between the CFRP laminate and the steel beam was not strong enough to resist the combination effect of the stress concentration and the opening force at the notch, bond failure initiated from the mid-span and the crack went through the whole bondline too rapidly to be noticed. As a result, a sudden bond failure happened before the beam went to plastic deformation stage.

The relationship between the load and the mid-span deflection of each specimen repaired with precured laminates is illustrated in Figure 4.15. Before bond failure happened, FRP laminates worked together with steel beam as a part of the tension flange. Consequently, both the bending stiffness and the load carrying capacity were effectively enhanced. For specimens with short bond length, the premature bond failure happened slightly earlier than that of specimens with full length bonding. It can be seen that the clamp assembly at the end of the laminate can not prevent the debonding of the FRP laminate since the bond failure initiated from the mid-span of the specimen.

The normalized mid-span deflections of specimens repaired with different laminates are compared in Figure 4.16. The deflection of specimen with two layers of UC laminates was effectively reduced to two thirds of that of the beam without repair. For specimen with SCH11 laminate, the deflection reducing effect was relatively limited.

It is notable that for damaged steel beams, the load carrying capacity in elastic region can be increased much more by adhesive bonding of FRP laminates compared to intact beams. The stiffness of damaged steel beam can be restored to a remarkable extent if no premature bond failure happens. Because the highly concentrated stress caused by material discontinuity, bond failure between FRP laminate and steel surface happened earlier before steel beam underwent plastic deformation.

4.3.3 Bond Failure Load

The relationships of load and deflection of specimens with one, three and five layers of FRP laminates are compared with that for the control beam as shown in Figure

4.17. It can be noted that all the retrofitted beams showed a strength enhancement over the control beam (specimen C1), and the strength enhancement increased with the number of laminate layers. Except for the control specimen, all the strengthened beams failed by debonding of the FRP laminates and after attaining a peak the load dropped suddenly and the load–displacement plot then after followed closely the response of the control specimen. Debonding failure occurred because of the highly concentrated interfacial stress at the end of the bondline. This level of stress concentration was dependent on the applied bending moment and the location where the FRP laminate terminated. The mechanism of bond failure and major parameters influencing the debonding load is discussed in detail in Chapter 6. The experimental results of specimens in the last group are listed in Table 4.6. Figure 4.18 shows the relationship between normalized deflection at bond failure and number of laminate layers. The normalized deflection D_N is defined as the ratio of the deflection at bond failure of the specimen with multi-layer laminates to that of the specimen with one layer of laminates. The deflection at bond failure was reduced as the number of laminate layers increasing. The comparison of bond failure loads and deflections of these specimens shows that for FRP strengthened steel beams, the load at bond failure decreases with the increase of the thickness of the FRP laminates when the other parameters are same.

The responses for specimens C5 and C6, bonded with one and three layers of FRP laminates with spew fillet at the end of the bondline, are compared with those of the specimens without spew fillet in Figure 4.19. The bond failure loads of specimens C5 and C6 were higher than those of specimens C2 and C3, which showed the spew fillet can increase the bond strength of FRP strengthened steel beams although the effect was not so remarkable. The strength of C5 was a little lower than that of C6. This was an anomaly because a thin laminate thickness was expected to give better strength enhancement. Further investigation has found that at the end of the

bondline, specimen C5 had an obvious cavity in the solidified epoxy at the end of the bondline, which was probably caused by the poor workmanship during the bonding and curing process.

The results of specimens with three different thicknesses of the adhesive are presented in Figure 4.20. The adhesive layer of specimen C7 had double thickness and the adhesive thickness of specimen C8 was five times of that of specimen C2. Based on the comparison of the relationships of load and deflection of these specimens, it is obviously notable that the thicker the adhesive layer is, the weaker the bond strength between the FRP laminate and the steel beam. This result was consistent with the experiential observations of FRP bonded concrete structures. To explain the effect of the adhesive thickness on the bond strength, a statistics-based theory was proposed by Adams and Peppiatt (1974). They said that thicker bondlines contain more defects such as voids and microcracks so that they tended to fail earlier than the thinner ones. Another explanation was given by Gleich et al. (2001). They showed that the interface stress concentration level increased as the bondline got thicker by conducting FE analysis on adhesive FRP joints with different adhesive thicknesses. Besides these two explanations, another reason for this effect should be attributed to the special stress state of the adhesive. Compared to material properties of the steel and FRP adherends, the stiffness and thickness of the interfacial adhesive is very small. When the adhesive undergoes large plastic deformation, the adherends are still under relatively small elastic deformation. Therefore, the deformation of the adhesive is under constraint of the adjacent adherends to some extent. It is clear that the thicker the adhesive layer is, the smaller this constraint effect. Therefore the joint with thicker adhesive tends to fail at smaller load level since there is less deformation-constraint effect.

Tapers were created at the ends of the FRP laminates used to strengthen steel beams for specimens C9 and C10. Adhesive spew fillet at the end of bondline was also created for specimen C10. The load-deflection curves for these specimens are compared with specimen without taper on the laminate end as shown in Figure 4.21. The deflection at bond failure of the specimen with taper on the FRP laminate was much larger than that of specimen without taper. It indicates that the taper at the end of FRP laminate can effectively increase the bond strength of FRP strengthened steel beam. For deflections of specimens with taper, there was only slight difference between the specimens with and without spew fillet. Therefore, the effect of spew fillet on the bond strength for specimens with taper was not significant. Same tapers were also created for the FRP laminates to strengthen steel beams of specimens C11 and C12. Different from the two specimens discussed above, these laminates with taper were reversed when bonded to the steel beam, which means instead of facing outward the taper on the laminate was toward the steel beam and bonded with epoxy as part of adherend interface. The effect of the so-called reversed taper on bond failure load is presented in Figure 4.22. The specimen with reversed taper at the end of bondline showed very promising effect on enhancing the bond strength of the strengthened steel beam. For specimen with reversed taper and spew fillet together at the end of the bondline, there was no bond failure was observed throughout the loading process. Based on all the discussions above and the comparison shown in Figure 4.23, it can be seen that among all the experimentally investigated measures the combination of reversed taper at the end of FRP laminate and spew fillet at the end of bondline was the most effective means to increase the bond strength of FRP bonded steel beam.

The load-deflection curves for specimens with different bond lengths are plotted in Figure 4.24. Four specimens with different bond lengths were investigated. The length of the bonded laminate was increase from 750 mm to 1450 mm. With the

length of the bonded FRP laminate increasing, the bond failure load of the strengthened beam was improved. The relationship between deflection at bond failure and bond length of FRP laminate is illustrated in Figure 4.25. The stress distributed in the end of the bondline played a significant role in determining the bond strength of FRP strengthened steel beams. For a beam strengthened with short FRP laminate, the stress level at the end of bondline was quite high since the large force in tension flange needed to be transferred into the laminate by means of deformation of the interfacial adhesive. The deformation of the adhesive mainly concentrated at the end of the bondline. Therefore, the stress level in the end of the bondline was very high in this case. For a beam bonded with relatively long laminate, the largely deformed zone was more widely and uniformly distributed along the bondline, thus the less the stress concentration level at the end of the bondline. Therefore for a FRP strengthened steel beam, the longer bond length means higher bond strength.

4.3.4 Failure Modes

For specimens with different configurations, the failure modes could be quite different. In the experimental program, totally five different failure modes were observed, i.e. i) steel beam yielding (SY), ii) steel yielding followed by splitting and debonding of FRP laminate (YS), iii) steel yielding following or followed by debonding of FRP laminate (DY), iv) fracture of steel beam (SF) and, v) debonding of FRP laminate followed by steel fracture (DF). The pictures of some failed specimens are shown in Figure 4.26. For intact steel beam strengthened with FRP laminate, yielding and plastic deformation of steel was most common failure mode as shown in Figure 4.26(a). For specimens bonded with full-length FRP laminate, debonding of laminate generally did not happen throughout the loading process. Due to the poor quality of bonding produced during the manufacture process,

debonding failure still happened for one specimen with full length laminate after the steel beam underwent large plastic deformation. For specimens with full-length SCH11 laminate, debonding happened together with splitting of the laminate along longitude direction. Since there was almost no connection between the neighboring fibers except the bond of the epoxy, the strength of the laminate in the transverse direction was very small. Therefore it was not abnormal for the splitting of FRP laminate even subjected to small load or deformation in transverse direction. For the beams strengthened with partial-length FRP laminate, debonding of laminate could happen before or after the yielding of the steel beam. The sequence of these two failure modes depends on the strengthening configuration of the specimen. For FRP strengthened damaged steel beam, the fracture of steel initiated from the damage was accompanied by the debonding of FRP laminate with the same initiation location. The bond failure for intact steel beams strengthened by FRP laminate generally initiated from the end of the bondline. For specimens without adhesive spew fillet at the end of the bondline, the debonding initiated from the end tip of the epoxy where the stress concentration level was the highest. For specimens with spew fillet at the end of the bondline, the debonding initiated from the epoxy close to the tip of the FRP laminate because the stress concentration level at the tip of the epoxy was dramatically reduced by the spew fillet.

4.3.5 Stress Distribution and Force Transfer Mechanism

In the experiment, strain gauges were mounted to both the steel beams and the surface of the FRP laminates to measure the strain distributions at different locations. Strain distributions on the front and back sides of the web along the depth of the beam at mid-span of specimen A6 are shown in Figures 4.27 and 4.28, respectively. It can be seen that on the front side, the absolute value of strain in the inner side of compression flange tended to increase faster compared to the strains in

other locations. Simultaneously, the strain in tension flange tended to increase slower. On the contrary, the strains in the corresponding locations on the other side had inverse increasing tendency. This difference was mainly caused by the transverse displacement of the beam. Because in the experiment there was no special measure to restrict the lateral movement of the beam, every specimen had more or less out-of-plane deflection under compression loading due to unavoidable initial asymmetry caused by the manufacture defects. A larger the compression load meant a higher deflection. As a result of this horizontal deflection, the strains distributed on the front and the back sides of the beam are different. It is indicated that bond and force transfer between steel and FRP laminate was not influenced by the horizontal deflection of the beam.

With the gradual increase of load, the position of the neutral axis of the steel beam could vary from time to time. With the external bonding of FRP laminates, the position and varying trend of the neutral axis of the steel beam could be changed. By interpolating the values of strains distributed in the web of the steel beam, the position of the neutral axis of each specimen can be found out. The distances from the neutral axis to the beam bottom of specimens with different laminate layers are compared in Figure 4.29. It can be found that the external bonding of FRP laminates had a remarkable influence on the position of the neutral axis of the beam. In elastic deformation region, the distance from the neutral axis to the beam bottom was reduced by 20 mm with six layers of FRP laminates bonded. In plastic region, the fast rise of neutral axis was also effectively restrained. Bonded with six layers of laminates, the offset of neutral axis from its original position throughout the loading was reduced from 50 mm to less than 20 mm.

As discussed earlier, under bending load the web bottom of the steel beam yielded first. Then, the yielding zone expanded upward gradually with the load increasing. Similar to the analysis above, the boundary of yield zone of each specimen can be found out by interpolating the values of strains at relevant locations on the web. In Figure 4.30, the differences of yield boundary development between specimens with various laminate layers are presented. For the beam without external bonding of FRP laminates, the steel web begins yielding when the bending moment was a little more than 80 kN. With the load going up, the boundary of yield zone extended more than 180 mm from the bottom of the beam. With FRP laminates bonded, the emergence of the yield area was postponed and the expanding of yield zone was restrained to a low pace. For the specimen with six layers of laminates, the boundary extension of yield area was almost reduced by 50%. The development of yielding boundary of specimens in the second group is shown in Figure 4.31. For the specimen with SCH11 laminate, the variation of the boundary of yield zone had little difference with that of the un-strengthened beam. For the specimen with UC laminate, the expanding of yield zone was effectively restrained.

The strain distributions in the laminate of specimen A6 are shown in Figure 4.32. It is found that the deformation of the steel beam had significant effect on the strain development in the FRP laminate. At the beginning of the loading stage, the profile of the strain followed a trapezoidal shape which matched moment distribution diagram of the steel beam very nicely. After the load passed the yielding moment around 60 kNm, the strain in the mid section increased very fast, especially those in-between the loading points. This apparently was caused by the local plastic deformation of the bottom flange of the steel beam. The comparison of stress development in beam bottom and FRP laminate of beam A6 is shown in Figure 4.33. The stress in the steel almost did not increase after yielding. Contrarily, the stress in FRP laminate increased much faster after steel yielding. This implies

significant transfer of tension force from the bottom of the steel beam to the external FRP laminate. The FRP laminate did not yield like steel so it continued to bear the force with the plastic deformation of steel increasing. This explains why the increase of the ultimate load-carrying capacity of the steel beam was more than that of the elastic flexural stiffness with only one layer of FRP laminate. In comparison with the sharp increase in the maximum moment area, the variation of strain in shear span, particularly in the vicinity of plate end was insignificant.

From the above discussion it can be seen that the strengthening effect of FRP laminates on flexural capacity of steel beams is realized by the tension transferred from steel beam to FRP. The advantage of high strength of FRP can be much better utilized when the steel beam undergoes plastic deformation. The FRP in the maximum moment zone is subject to the highest tensile stress. Therefore, it is advisable to applied more layers of FRP laminates under the maximum moment zone of the beam in strengthening application.

Experimental Study of FRP Strengthened Steel Beams under Static Loading

Table 4.1 Details of specimens in group A (intact beams)

Specimen	Laminate	Epoxy	Number of layers	Fiber	Bond length (mm)	clamp
A1	-	-	-	-	-	-
A2	SCH11	S	2	Carbon	1500	No
A3	SCH41	S	1	Carbon	1500	No
A4	SEH51	S	1	Glass	1500	No
A5	UC	TC	1	Carbon	1500	No
A6	UC	TC	1	Carbon	1500	Yes
A7	UG	TC	1	Glass	1500	No
A8	-	-	-	-	-	-
A9	SCH11	S	2	Carbon	1500	No
A10	SCH11	S	4	Carbon	1500	No
A11	SCH11	S	6	Carbon	1500	No

Note: “Clamp” refers to the mechanical clamp system applied at the end of the bondline; for control beams A1 and A8 the strengthening parameters are not applicable; “Yes” indicates the application of clamp system; “No” indicates no clamp system application.

Table 4.2 Details of specimens in group B (damaged beams)

Specimen	Laminate	Epoxy	Number of layers	Bond Length (mm)	Clamp
B1	-	-	-	-	-
B2	SCH11	S	1	1500	No
B3	SCH41	S	1	1500	No
B4	UC	TC	1	900	No
B5	UC	TC	1	900	Yes
B6	UC	TC	1	1500	No
B7	UC	TC	2	1500	No

Note: “Clamp” refers to the mechanical clamp system applied at the end of the bondline; for control beam B1 the strengthening parameters are not applicable; “Yes” indicates the application of clamp system; “No” indicates no clamp system application.

Experimental Study of FRP Strengthened Steel Beams under Static Loading

Table 4.3 Details of specimens in group C (beams with different bond parameters)

Specimen	Laminate	Epoxy	Number of FRP layers	Epoxy thickness (mm)	Bond length (mm)	Taper	Fillet
C1	-	-	-	-	-	-	-
C2	UC	TC	1	0.5	700	No	No
C3	UC	TC	3	0.5	700	No	No
C4	UC	TC	5	0.5	700	No	No
C5	UC	TC	1	0.5	700	No	Yes
C6	UC	TC	3	0.5	700	No	Yes
C7	UC	TC	1	1	700	No	No
C8	UC	TC	1	2.5	700	No	No
C9	UC	TC	1	0.5	700	Yes	No
C10	UC	TC	1	0.5	700	Yes	Yes
C11	UC	TC	1	0.5	700	Re	No
C12	UC	TC	1	0.5	700	Re	Yes
C13	SCH41	S	1	0.5	700	No	No
C14	-	-	-	-	-	-	-
C15	UC	TC	1	0.5	750	No	No
C16	UC	TC	1	0.5	850	No	No
C17	UC	TC	1	0.5	1050	No	No
C18	UC	TC	1	0.5	1450	No	No

Note: “Taper” refers to the gradually thinner end of the laminate created before bonding; “Fillet” refers to the extrusion of the solidified epoxy at the end of the bondline; for control beams C1 and C14 the strengthening parameters are not applicable.

Experimental Study of FRP Strengthened Steel Beams under Static Loading

Table 4.4 Experimental results for specimens in group A

Specimen	Maximum load (kN)	Increase (%)	Load at $L/d = 360$ (kN)	Increase (%)	Failure mode
A1	196.5	0.0	101.5	0.0	SY
A2	231.7	17.9	120.5	18.7	YS
A3	232.4	18.3	117.5	15.8	SY
A4	211.5	7.6	107.5	5.9	SY
A5	231.3	17.7	120.3	18.5	SY
A6	230.9	17.5	118.2	16.5	SY
A7	218.2	11.0	110.2	8.6	SY
A8	472.0	0.0	349.9	0.0	SY
A9	484.1	2.5	358.1	2.3	SY
A10	511.2	8.3	360.8	3.1	SY
A11	515.8	9.3	366.1	4.6	DY

Note: SY- Steel beam yielding; YS- Steel yielding followed by splitting and debonding of FRP laminate; DY- Steel yielding following or followed by debonding of FRP laminate.

Table 4.5 Experimental results for specimens in group B

Specimen	Maximum load (kN)	Increase (%)	Load at $L/d = 360$ (kN)	Increase (%)	Failure mode
B1	272.2	0.0	180.7	0.0	SF
B2	281.4	3.3	184.8	2.2	DF
B3	277.6	2.1	219.6	21.3	DF
B4	312.3	14.7	234.5	29.7	DF
B5	285	4.8	226	24.9	DF
B6	409.9	50.7	240.2	32.7	DF
B7	437.8	61.0	269.9	49.1	DF

Note: SF- Fracture of steel beam; DF- Debonding of FRP laminate followed by steel beam fracture.

Table 4.6 Experimental results for specimens in group C

Specimen	M (kNm)	Increase (%)	D_0 (mm)	Failure mode
C1	48.7	-	-	SY
C2	51.0	4.7	20.5	DY
C3	55.0	13.0	18.7	DY
C4	56.9	17.0	14.6	DY
C5	50.6	4.0	25.2	DY
C6	57.3	17.8	26.1	DY
C7	50.5	3.8	19.0	DY
C8	49.2	1.1	12.3	DY
C9	50.3	3.5	28.1	DY
C10	53.3	9.6	30.3	DY
C11	52.0	6.9	33.1	DY
C12	54.9	12.9	-	SY
C13	43.1	-	8.2	DY
C14	45.3	-	-	SY
C15	46.1	1.7	17.9	DY
C16	47.9	5.7	21.0	DY
C17	50.3	10.9	33.4	DY
C18	52.8	16.6	-	SY

Note: M - Bending moment; D_0 - Mid-span deflection at bond failure; SY- Steel beam yielding; DY- Steel yielding following or followed by debonding of FRP laminate; For specimens without strengthening C1 and C14 and specimen C13 with debonding happened earlier than steel beam yielding, the increase of bending moment is not applicable; for specimens without strengthening and specimens without bond failure C12 and C18, the mid-span deflection at bond failure is not applicable.

Figure 4.1 Cross-section dimension of steel beam

Figure 4.2 Schematic of typical specimen configuration

Experimental Study of FRP Strengthened Steel Beams under Static Loading

Figure 4.3 Configuration of steel beams

Figure 4.4 Plan view of clamp assembly

Experimental Study of FRP Strengthened Steel Beams under Static Loading

(a) Grinding and cleaning

(b) Applying epoxy

(c) Fixing CFRP strip

(d) Pressing

(e) Removing excessive epoxy

(f) Curing

Figure 4.5 Procedure of bonding FRP laminates to steel beams

Figure 4.6 Schematic of test setup

Figure 4.7 Picture of test setup

Experimental Study of FRP Strengthened Steel Beams under Static Loading

(a)

Figure 4.8 Strain gauge locations for typical specimens in (a), (b) group A and (c) group B

Figure 4.9 Relationship between moment and mid-span deflection of specimens strengthened with wet lay-up laminates

Figure 4.10 Relationship between moment and mid-span deflection of specimens strengthened with precured laminates

Figure 4.11 Comparison of deflections of specimens with different fibers

Figure 4.12 Relationship between moment and mid-span deflection of specimens with different layers of laminates

Figure 4.13 Comparison of load increases for specimens with different layers of laminates

Figure 4.14 Relationship between moment and mid-span deflection of specimens repaired with wet lay-up laminates

Figure 4.15 Relationship between moment and mid-span deflection of specimens repaired with precured laminates

Figure 4.16 Comparison of normalized mid-span deflections of specimens repaired with different laminates

Figure 4.17 Relationship between moment and mid-span deflection of specimens with different layers of laminates

Figure 4.18 Relationship between number of laminate layers and normalized debonding deflection

Figure 4.19 Relationship between moment and mid-span deflection of specimens with and without fillet

Figure 4.20 Relationship between moment and mid-span deflection of specimens with different epoxy thicknesses

Figure 4.21 Relationship between moment and mid-span deflection of specimens with and without taper

Figure 4.22 Relationship between moment and mid-span deflection of specimens with and without reversed taper

Figure 4.23 Comparison of normalized debonding deflections of specimens with different bond end details

Figure 4.24 Relationship between moment and mid-span deflection of specimens with different bond lengths

Figure 4.25 Relationship between bond length and normalized debonding deflection

Experimental Study of FRP Strengthened Steel Beams under Static Loading

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(a)

(b)

(c)

(d)

(e)

(f)

Figure 4.26 Failure modes of FRP strengthened steel beams: (a) steel beam yielding (b) debonding of FRP laminate from the end (c) debonding and splitting of FRP laminate (d) debonding of FRP laminate from damage zone (e) debonding from the end of epoxy (f) debonding from the end of FRP laminate

Figure 4.27 Strain distribution along front centerline of beam A6

Figure 4.28 Strain distribution along back centerline of beam A6

Experimental Study of FRP Strengthened Steel Beams under Static Loading

Figure 4.29 Distance from neutral axis to bottom of specimens with different laminate layers

Figure 4.30 Development of boundaries of yield zone of specimens with different laminate layers

Figure 4.31 Development of boundaries of yield zone of specimens in group B

Figure 4.32 Strain distribution along the FRP laminate of beam A6

Figure 4.33 Comparison of stress development in beam bottom and FRP laminate of beam A6

CHAPTER 5

FLEXURAL BEHAVIOR OF FRP STRENGTHENED STEEL BEAMS

5.1 INTRODUCTION

From the experimental results presented in the preceding chapter, it is known that external bonding of FRP materials can increase the elastic stiffness of a steel beam as well as the load carrying capacity. The following sections present a proposed design philosophy and analysis procedure that can be used to design the FRP strengthening for steel flexural members to achieve a desired increase of flexural strength under static loading.

The analysis of flexural strength of the strengthened beam was based on the assumption that bond failure was completely prevented. Since debonding of FRP laminate from substrate is a very important issue for FRP strengthened steel structures, this failure mode was not considered in current analysis of flexural capacity and the relevant investigations are presented in detail in next chapter. The flexural capacity without bond failure and the bond failure load were supposed to be considerate separately in the design procedure of the FRP strengthened steel flexural members.

A moment-curvature analysis was used to develop the equations of flexural capacity of FRP strengthened steel beams. This analysis was based on the cross-sectional geometry as well as the stress-strain behavior of the constituent materials. The analysis procedure of flexural stiffness, load carrying capacity and load-deflection

relationship is presented. Besides analytical solutions for flexural capacity, FE modeling was carried out to investigate the stress/strain distributions in FRP strengthened steel beams. A parametric study of effect of thickness, tensile modulus and bond length of FRP laminates on flexural stiffness and strength was carried out. An optimal thickness of FRP laminate for full length strengthening was found out.

5.2 ASSUMPTIONS

FRP strengthened steel beam is a composite structure which is composed three components with completely different material properties. The analysis of flexural strength of steel beam was complicated by the attachment of additional epoxy and FRP laminate. To make the analysis simple and efficient enough without loss of validity, the analysis of the stress-strain state and flexural capacity of a FRP strengthened steel beam subject to bending moments was carried out on the basis of the following main assumptions:

1. There is no slip between the steel beam and the FRP laminate, i.e. there is no bond failure happened throughout the bending process;
2. The elastic-perfect plastic stress–strain relationship is adopted as the material constitutive law for steel beam;
3. Plane sections remain plane when the beam is subjected to a bending moment;
4. The effect of shear force on deflection of the beam is negligible;
5. There is no variance of axial stress through the thickness of FRP laminate;
6. The thickness and the contribution of the adhesive layer to the stiffness of the strengthened beam are negligible.

Since the plastic deformation of steel beam is be involved in the analysis, the second assumption is different from that used in most other existing analyses in

which plastic deformation is not considered. Debonding of the FRP laminate is discussed separately in the chapter of bond failure model therefore not considered in current flexural analysis. For common low carbon steel material, the extent of plastic deformation is quite large after the yield of steel. Therefore, the second hypothesis is thought to be a reasonable simplification of material constitutive relationship for the steel beam. The third and forth hypotheses are well known and established in classic material mechanics theories. The fifth assumption allows for the FRP laminate to be assimilated to one single fiber, at the same level of the steel beam fiber to which it is bonded. The last hypothesis is justified by the very small value of the Young's modulus of the adhesive material with respect to that of the steel beam, in addition to the relatively small thickness of the adhesive layer.

5.3 ELASTIC CAPACITY

External bonding of FRP laminates can increase the elastic stiffness of a steel beam therefore reducing the elastic strain in the tension flange of the beam as compared to an un-strengthened beam at the same load level. Due to these two effects, the load capacity of a steel beam can be increased using adhesively bonded FRP materials. This section presents an analytical procedure that can be used to calculate flexural capacity of FRP strengthened steel beams.

5.3.1 Elastic Moment

For a FRP strengthened steel beam, the maximum elastic moment is reached when the extreme fiber of the steel beam reaches the yield strain. By analyzing the strain compatibility and force equilibrium of the critical section of the beam, the maximum elastic moment of the beam can be found out. To facilitate the calculation of the section property of the beam, the equivalent stiffness transformation method

was used to convert the original FRP section to a virtual steel section which has the same contribution of flexural stiffness to the beam as the FRP section. The schematic of the equivalent stiffness transformation is illustrated in Figure 5.1. After the transformation, the virtual steel section had the same elastic modulus as that of the steel beam. To make the stiffness of the whole section unchanged, the width of the virtual steel section was determined by the following calculation:

$$b' = \frac{E_f}{E} b_0 \quad (5.1)$$

where b' is the width of the virtual FRP section, E_f is the elastic modulus of the FRP laminate, E is the elastic modulus of the steel beam and b_0 is the width of the width of the original FRP laminate section. The thickness of the virtual FRP section remained same as that of the original FRP laminate section. Since the change of thickness of additional strengthening layer could result in a different plastic modulus of inertia after yielding of the steel beam, the conversion of section area would have been invalid if the thickness instead of width of the section was selected to be changed.

After the transformation, the whole section had a uniformed elastic modulus. Therefore, the flexural stiffness can be easily found out and consequently the design value of elastic bending moment.

For example, for a steel beam with cross section as shown in Figure 5.2, the second moment of area of the beam is I . The beam has an I-section and the thickness and the width of the flange are t_f and b , respectively. To simplify the calculation, the thickness of the web was assumed to be very small compared to that of the flange and its contribution to the flexural stiffness was deemed to be negligible. The depth of the beam was assumed to be very large compared to the thickness of the flange. A FRP laminate with same width as that of the flange was bonded to the bottom of

the beam. The thickness of the laminate was half of that of the flange and the elastic modulus of laminate was twice of that of the steel. By using the equation of equivalent stiffness transformation, the FRP section can be converted to a virtual steel section with a width of $2b$. For the original steel beam, the flexural stiffness is

$s = EI$ and the maximum elastic moment is $M_e = \frac{\sigma_y I}{y}$, where σ_y is the yield stress

of the steel and y is the distance from the bottom extreme fiber to the neutral axis of the beam. For the strengthened beam, the neutral axis of the section moves downwards and the distance from the steel extreme tension fiber to the neutral axis is $y' = 2y/3$. The new second moment of area I' is increased to $I' = 4I/3$.

Accordingly, the flexural stiffness of the strengthened beam is $s' = 4EI/3$ and the

maximum elastic moment is $M'_e = \frac{\sigma_y I}{y}$. It can be seen that the flexural stiffness has

been increased effectively by one third of the original stiffness after strengthening. However, the maximum elastic moment has not been enhanced. This is because the movement of the neutral axis has resulted in an earlier yield of the extreme fiber and thus has counteracted the effect of FRP laminate on elastic moment improvement. This example showing how the elastic modulus and moment can be influenced by the bonding of FRP laminates is just a specific case used for illustration of the analysis process. Therefore, all the specific relationships between the parameters may not be applicable to steel beams with different section properties.

5.3.2 Elastic Deflection

With the equivalent stiffness transformation, the calculation procedure of elastic deflection of a FRP strengthened steel beam under flexural load is just as same as that of a common steel beam. For instance, the maximum deflection of the un-strengthened beam mentioned above is $D = ML^2/(8EI)$, where L is span of the

beam. For the FRP strengthened steel beam, the maximum deflection is $D' = 3ML^2 / (32EI)$. Since the beam is under elastic deformation, the maximum deflection of the beam increases linearly with the flexural load. That means moment-deflection curve of the beam under elastic deformation is a straight line.

5.4 ULTIMATE CAPACITY

In the ultimate limit states design method, the ultimate load carrying capacity of the beam is required. Compared to the calculation of elastic load carrying capacity, the analysis procedure for ultimate load will be much more complicated on account of the nonlinearity of the steel material. The stiffness of the strengthened beam will decrease when the extreme fiber of the steel beam start yielding. The flexural stiffness of the beam will not be uniformly distributed throughout the loading process any more. Since in real flexural members the external moment can not be constant along the length of the beam, the flexural stiffness after yielding will no longer be constant along the beam as well. To analyze the moment and deflection of FRP strengthened beam under ultimate limit states, the strain compatibility and force equilibrium conditions were utilized. Although the methodology used in analysis is not new, the analytical solution of FRP strengthened steel beams under plastic deformation is not available yet. The calculation was based on the non-linear stress-strain behavior of the steel as well as the linear behavior of the FRP laminates.

5.4.1 Ultimate Load

As a result of the fundamental differences in the behavior of the steel and the strengthening FRP laminate, an elastic-plastic analysis is required, rather than a fully plastic analysis. The steel will deform in a linear elastic manner up to the limit

of proportionality, after which it will deform plastically to failure at a large ultimate strain, whereas FRP laminate will deform elastically all through to failure. The ultimate load carrying capacity of a FRP strengthened steel beam can be dependent on the limit values of strains of either the FRP laminate or the steel beam.

The elastic-plastic analysis is based on the cross-sectional geometry as well as the stress-strain behavior of the constituent materials using the principal of strain compatibility. Under the assumption that cross section plane remaining plane throughout deformation, the equilibrium of resultant forces at the cross-section can be utilized. As shown from Figure 5.3 to Figure 5.5, the cross-section of the strengthened beam can be broken down into different levels corresponding to the plastic compression steel, the elastic compression steel, the elastic tension steel, the plastic tension steel and the FRP laminate at the bottom of the cross-section. The resultant forces of the different components of the cross-section can be calculated by the integration of the corresponding stress profile. In Figure 5.3, the steel is still within elastic deformation stage, therefore there are only three resultant forces, namely elastic compression F_c , elastic tension F_t and tension in FRP laminate F_f . The equilibrium of resultant forces at the cross-section is as follows,

$$F_f + F_t = F_c \quad (5.2)$$

With the load increasing, the top of the compression steel will yield first as shown in Figure 5.4. In this case, the compression force is composed of two parts, i.e. the plastic compression F_{cp} and elastic compression F_{ce} . Consequently, the equilibrium of the cross-section forces has the following expression:

$$F_f + F_t = F_{cp} + F_{ce} \quad (5.3)$$

If the rupture strain of the FRP laminate is large enough, the bottom tension steel will also undergo plastic deformation before failure. As illustrated in Figure 5.5, in addition to the elastic tension F_{te} , there comes another tension component, which is plastic tension F_{tp} . Accordingly, the equilibrium equation will change to:

$$F_f + F_{tp} + F_{te} = F_{cp} + F_{ce} \quad (5.4)$$

Based on the strain compatibility and the material constrictive relationship, each resultant force in the above equations can be calculated by the integration of the corresponding stress profile and then expressed as a function composed of two independent variables, i.e. the strain at the extreme fiber of the beam ε_0 and the distance from the extreme fiber to the neutral axis of the beam y_0 . In other words, any resultant force F_i can be written as the following expression:

$$F_i = f_i(\varepsilon_0, y_0) \quad (5.5)$$

For example, for a FRP strengthened steel beam as shown in Figure 5.6, the compression plastic force is:

$$F_{cp} = \int_{y_l}^{y_l+y_p} \sigma(y)b(y)dy \quad (5.6)$$

where, y is the distance from the neutral axis to the small element, y_l is the distance from the neutral axis to the bottom of the plastic zone, y_p is the depth of the plastic zone, $\sigma(y)$ is the stress in the small element and $b(y)$ is the width of the element. Based on the strain compatibility at the cross-section as well as the elastic-perfect plastic stress-strain relationship for the steel material, the compression plastic force can be then rewritten as the following equation:

$$F_{cp} = \left[h - \left(1 + \frac{\sigma_y}{E\varepsilon_0}\right)y_0 \right] b\sigma_y \quad (5.7)$$

where h is the depth of the steel beam, σ_y is the yield stress of the steel and b is the width of the steel beam. Similarly, the other resultant forces can also be expressed as functions with respect to these two variables:

$$F_{ce} = \frac{\sigma_y}{2E\varepsilon_0} y_0 b\sigma_y \quad (5.8)$$

$$F_{tp} = \left(1 - \frac{\sigma_y}{E\varepsilon_0}\right) y_0 b\sigma_y \quad (5.9)$$

$$F_{te} = \frac{\sigma_y}{2E\varepsilon_0} y_0 b \sigma_y \quad (5.10)$$

$$F_f = E_f \varepsilon_0 b t \quad (5.11)$$

where E_f is the elastic modulus of FRP laminate, which is a constant value. By substituting the expression of each resultant force, i.e. equation 5.5 into the cross-section equilibrium equation together with necessary algebraic transformation, the distance from extreme strain fiber to the neural axis can be expressed as a function with respect to the extreme fiber strain as follows:

$$y_0 = f_y(\varepsilon_0) \quad (5.12)$$

With this equation, the position of the neutral axis can be found out if the extreme strain is known. Consequently, the moment capacity of the section can be determined by the combination of each multiplication of the resultant force and the corresponding distance d_i as follows:

$$M = \sum F_i d_i \quad (5.13)$$

Similar to the resultant forces, the distances can also be expressed as functions with respect to the two variables. In this example, the equations for five distances are as follows:

$$d_{cp} = \frac{1}{2} \left[h + \left(\frac{\sigma_y}{E\varepsilon_0} - 1 \right) y_0 \right] \quad (5.14)$$

$$d_{ce} = \frac{2\sigma_y}{3E\varepsilon_0} y_0 \quad (5.15)$$

$$d_p = \frac{1}{2} \left(1 + \frac{\sigma_y}{E\varepsilon_0} \right) y_0 \quad (5.16)$$

$$d_{te} = \frac{2\sigma_y}{3E\varepsilon_0} y_0 \quad (5.17)$$

$$d_f = y_0 + \frac{t}{2} \quad (5.18)$$

By combining equations 5.7~5.11 with equation 5.4 together with necessary algebraic transformations, the neutral axis depth can be expressed as:

$$y_0 = \frac{1}{2} \left(h - \frac{E_f t}{\sigma_y} \varepsilon_0 \right) \quad (5.19)$$

By substituting equations 5.7~5.11 and 5.14~5.19 to equation 5.13, the ultimate moment capacity of the section can be found out with the known ultimate design value of strain. The ultimate value of strain is dependant on the material properties of the steel beam as well as those of the FRP laminate. In this study, the ultimate strains for various FRP laminates used are from 1.1% to 2%. The ultimate strain for the steel is more than 2%. Therefore, the maximum design value can be used in this study is 1.1%. In engineering application or real design process, this value could be smaller with the incorporation of safety factors. Since the deflection of the specimens could be very large when the ultimate capacity of the specimens is reached, in the experiments, the real ultimate strength of the specimens without bond failure was not obtained since the test was stopped before the ultimate deflection was reached on consideration of the safety during the test process. Theretofore, the analytical procedure for the moment capacity of the FRP strengthened steel beam under ultimate limit states has been illustrated. The essential of the analysis is composed of the following steps. First, each resultant force of the cross-section is found out and expressed as a function with respect to location of neutral axis and extreme fiber strain. Second, similarly to the first step, the distance of each resultant force to the neutral axis is expressed as a function with respect to the distance to neutral axis and extreme strain. By substituting the resultant forces to the equilibrium equation, the neutral axis depth can be found out as a function of extreme strain. Once the location of the neutral axis has been determined, the moment capacity can be calculated by multiplication of forces and the associated distances from the centroid of the force block to the neutral axis.

5.4.2 Load-Deflection Relationship

From the analysis of ultimate moment capacity of the FRP strengthened steel beam, it can be seen that when the beam undergoes plastic deformation, the location of neutral axis can be determined when the extreme fiber strain is known and vice versa. That means, there is a one to one correspondence relationship between these two variables. It is also noticeable that the increase of plastic moment is always corresponding to the increase of extreme fiber strain and the concurrent decrease of distance from the neutral axis to extreme fiber of the beam. Based on the assumption of plane sections remaining plane and the geometry of the cross-section, the curvature can be determined as,

$$\frac{1}{\rho} = \frac{\varepsilon_0}{y_0} \quad (5.20)$$

where ρ is the radius of curvature. Since there is a one to one correspondence relationship between location of neutral axis, extreme fiber strain and moment, the following equation can be obtained:

$$\frac{\varepsilon_0}{y_0} = f_{\rho}(M) \quad (5.21)$$

The moment–curvature behavior of a given cross-section is determined based on the extreme fiber strain together with the neutral axis depth. Substituting this equation to 5.20, the relationship between curvature and moment of the section is determined:

$$\frac{1}{\rho} = f_{\rho}(M) \quad (5.22)$$

For practical engineering structures, the magnitude of deflection is generally very small compared to that of the span of the flexural member and the deflection of the beam can be determined by integration of the curvature:

$$D(x) = \iint \frac{1}{\rho(x)} d^2x \quad (5.23)$$

where x is the distance along the beam axis from any beam element to a selected supporting point of the beam. If the moment along the beam is expressed by the following equation:

$$M = f_m(x) \quad (5.24)$$

then from equations 5.22 and 5.23, the deflection of the beam can be determined as follows:

$$D(x) = \iint f_\rho[f_m(x)] d^2x \quad (5.25)$$

Based on the calculation of plastic deflection, the load-deflection relationship of a FRP strengthened steel beam under plastic deformation can be determined. The load-deflection relationship of a FRP strengthened steel beam throughout the loading process can be determined by combination of the load-deflection relationships under both elastic and plastic deformation.

5.5 EVALUATION OF ANALYTICAL MODEL

Validation of the analytical model was conducted by comparing the results of the FRP strengthened steel beams tested in the experimental program with the analytical results. The material properties required by the analysis were taken from results of the material tests as indicted in Chapter 4.

5.5.1 Load Capacity

The comparison between experimental and analytical results of load capacities of different specimens is shown in Table 5.1. It can be seen that the analytical load capacities of the FRP strengthened steel beams agreed well with the experimental

results. The material behavior of the steel in the analytical model was assumed to have an elastic perfect plastic stress-strain relationship. This simplification caused relatively larger difference of ultimate loads between analytical and experimental results, since the steel material tested in the experiments exhibited strain hardening behavior after the steel yielded. In practice, large deflections of flexural members in structural engineering are generally restricted to certain extent of $L/120$ or less and the steel members do not undergo large plastic deformations under the design load. Therefore this simplification of material property had no significant influence on the validation of the analytical model.

5.5.2 Deflection

The experimental and analytical load-deflection relationships of steel beams strengthened with FRP laminates are compared in Figure 5.7. The analytical result was found to agree well with the experimental result. The analytical deflection in elastic deformation stage was almost as same as that of experimental result. In the plastic deformation stage, the analytical deflection was slightly higher than that measured in the experiment as shown in Figure 5.8. Since the deformation from the analytical result was higher than that from experiment, the analytical result is thought to be conservative for the calculation of flexural capacity of FRP strengthened steel beams.

5.6 FE MODELING

Closed form solutions of the stress distribution for general configurations of FRP strengthened steel beams are not easily derivable. In these cases numerical methods must be adopted to investigate the stress distribution. In this work a finite element model was developed to evaluate the stresses in the strengthened beam. There was

no newly developed material model or new theoretical derivation of a new element used in the FE analysis. The FE model was also used for the parametric study in the next section.

5.6.1 Modeling Methodology

The type of finite elements to be used for modeling the FRP-steel structures will determine the layout of the model and the mesh density as well as the accuracy and efficiency of the analysis. As mentioned earlier, modeling thin walled steel structures using shell elements has the advantage of both good precision and high efficiency. Result of failure load obtained by using shell elements in FE analysis was proved to be accurate enough and there was almost of no difference with that of solid element strategy according to Taylor (1996). In the ABAQUS program, element type S4R is a general-purpose three-dimensional shell element. It is a 4-node doubly curved shell element, which has reduced integration with hourglass control. Each node has 6 degrees of freedom: 3 translational and 3 rotational. The stress-strain relationships of FRP and steel materials used in the FE analysis are illustrated in Figure 5.9. The nonlinear and the linear material curves were used for the steel and the FRP laminates respectively.

Due to the symmetry of geometry and loading conditions of the specimen, the steel beam can be modeled with only a quarter of the whole structure. On consideration of the display effect when the result is compared with experimental result, the symmetry about the web is neglected in the modeling and one half of the specimen is modeled as illustrated in Figure 5.10. The position of vertical load applied on the top of the flange of the steel beam is as the same as that in the experiment. The simply supported boundary condition is applied to the beam end. The symmetrical boundary conditions are applied to the mid-span cross-section.

The adhesive layer is not modeled in the FE analysis, since the bond failure is not considered in this study and there is assumed no relative movement between the FRP laminate and the steel beam. The steel beam and the FRP laminate are modeled as separate parts and linked together in the analysis. At the material interface between the steel beam and the FRP laminate, there exists a pairs of nodes at each identical location, one belonging to the adherend, the other to the adhesive. All degrees of freedom of the corresponding nodes are constrained to identical displacements to simulate the perfect bond. In the analysis, the geometric nonlinearity is considered by employing large-displacement formulation and Newton method is used for the matrix solver. The value of 1E-5 is adopted as convergence relevant tolerance.

5.6.2 Evaluation of FE Model

To examine the convergence of the FE Procedure, models with different meshing densities were developed in the FE analysis. The results with increasing number of element in FE model are shown in Figure 5.11. It can be seen that the flexural capacities of the models converged at the point of 15000 elements. Therefore, it can be concluded that the mesh of the fourth model is dense enough to be used in the subsequent analysis.

The results of FE analysis were compared with the experimental results. The load-deflection relationships for steel beams strengthened with different FRP laminates between experimental result and numerical analysis are compared in Figure 5.12. It is clear that the results of FE analysis agreed well with the corresponding experimental results.

The comparison of deformed shapes of specimens between experimental and FE results are illustrated in Figure 5.13. The deformation shapes of the flange of the steel beam from the FE analyses and the experiment were similar. The validity of the FE model has been further verified.

5.6.3 Stress/Strain Distribution

The stress distribution in the FRP strengthened steel beam is illustrated by means of stress contour steel beam and the FRP laminate as shown in Figure 5.14. The top and the bottom areas of the web and flanges in the middle of the beam yielded first. For the FRP laminate, the maximum stress appeared in the center area as well. As shown in Figure 5.15, the stress distribution in the FRP laminate longitudinal centerline along the bond length of the numerical model agreed well with that of the tested specimen. There was almost no stress variation within the pure moment region of the beam. The stress developments in the FRP laminate and the steel bottom flange with the increasing load is plotted in Figure 5.16. The stresses in the FRP laminate and the steel bottom flange increased proportionally with the increase of load before yielding. After yielding of the steel beam, the stress in the FRP laminate increased more rapidly, which was similar to the phenomenon observed in the experiment, indicating the high strength property of the FRP laminate has been more effectively utilized.

5.7 PARAMETRIC STUDY

The effects of the following parameters on the flexural capacity of FRP strengthened steel beams were investigated by using the FE model: i) the thickness of the FRP laminate, ii) tensile modulus of FRP laminate and, iii) bond length. The

assumptions made were that there was a perfect bond between the FRP laminate and steel beam through the whole loading process.

5.7.1 Effect of FRP Thickness

The relationship between number of layers of FRP laminates and increase of the flexural capacity Δ is shown in Figure 5.17. It is notable that the relationship between the increase of elastic flexural stiffness (S_e) and the number of layers of FRP laminates was almost linear. The increase in the ultimate strength (P_u) was much higher than the flexural stiffness when the FRP thickness was small. With the number of FRP layers going up, the difference between the stiffness and strength increases became less noticeable. When the FRP laminates were more than six layers, the increase of flexural stiffness was more than that of ultimate strength. It is apparent that there exists an optimal value of number of FRP layers that can make the steel beam get same increases of flexural stiffness and ultimate strength. The relationship between number of FRP layers and maximum strain in the FRP laminates is illustrated in Figure 5.18. It can be noted that the more the number of layers of FRP laminates, the lower the maximum strain in the laminate was. It indicates that it is not economical to use many layers of FRP laminates to strengthen steel beams since the tensile strength of the FRP can not be effectively utilized.

5.7.2 Effect of Tensile Modulus of FRP

The relationship between tensile modulus of FRP laminates and increase of the flexural capacity is shown in Figure 5.19. Similar to the effect of the number of FRP layers, FRP laminates with lower tensile modulus increased the ultimate strength of the steel beam much higher than the flexural stiffness. When the tensile modulus of FRP laminate was more 1000 kN/mm², the increase of flexural stiffness

went beyond that of ultimate strength of the steel beam. The relationship between tensile modulus of FRP laminates and the maximum strain ε_{max} in the FRP is explained in Figure 5.20. With the tensile modulus increasing, the maximum strain in the FRP went down.

5.7.3 Effect of Bond Length

Besides thickness and tensile modulus of FRP laminates, bond length is also an important parameter for FRP strengthened steel beams. In Figure 5.21, the relationship between the normalized bond length (l_0) and the increase of the flexural capacity is illustrated. When the bond length increased from 0.2 to 0.7, the increase of the ultimate strength of the beam was more prominent compared to that of the flexural stiffness. When the bond length was more than 0.7, the increases of both the stiffness and the strength became unnoticeable. The relationship between bond length and maximum strain in the FRP laminate is illustrated in Figure 5.22. Similar to the tendency of strength increase, the maximum strain in the FRP laminate increased very fast when the bond length was less than 0.7. The strain almost kept constant when the bond length was more than 0.7. It indicates that when the bond length has reached a certain value, further increase of the bond length is almost meaningless if the flexural capacity of the strengthened beam is concerned.

5.7.4 Concluding Remarks

Based on the parametric study of the FRP strengthened steel beams under static flexural loading, the impact of thickness of the FRP laminate, tensile modulus of FRP laminate and bond length on the flexural strengthening effect has been presented. The following conclusions can be drawn:

- 1) There is an optimal thickness of FRP laminate when the FRP modulus is constant and vice versa. When this optimal value is not reached, the increase of ultimate flexural strength is higher than that of elastic flexural stiffness. When this optimal value is surpassed, the increase of ultimate flexural strength is lower than that of elastic flexural stiffness.
- 2) The increase of the ultimate strength was more prominent compared to that of the flexural stiffness when the relative bond length is less than 0.7. When the bond length is more than 0.7, the increases of both stiffness and strength are relatively unnoticeable.
- 3) It is not economical to use FRP laminates with larger thickness or modulus for strengthening when the optimal value is reached. It is also not efficient to used very long bond length (larger than 0.7 in this study) for flexural strengthening.

Table 5.1 Comparison between experimental and analytical results

Specimen	Ultimate load	Analytical result	Difference (%)	Load at $L/d = 360$	Analytical result	Difference (%)
A1	196.5	207.1	5.4	101.5	108.9	-7.3
A2	231.7	216.9	-6.4	120.5	114.3	-5.1
A3	232.4	213.8	-8.0	117.5	112.6	-4.2
A4	211.5	209.5	-0.9	107.5	110.2	2.5
A5	231.3	224.2	-3.1	120.3	118.4	-1.6
A6	230.9	224.2	-2.9	118.2	118.4	0.2
A7	218.2	210.8	-3.4	110.2	110.9	0.6
A8	472.0	457.3	-3.1	349.9	344.7	-1.5
A9	484.1	471.2	-2.6	358.1	349.5	-2.4
A10	511.2	484.9	-5.1	360.8	354.2	-1.9
A11	515.8	497.7	-3.5	366.1	358.7	-2.0

Figure 5.1 Schematic of equivalent stiffness transformation

Figure 5.2 Section view of strengthened beam (a) without strengthening (b) bonded with FRP laminate (c) equivalent stiffness transformation

Figure 5.3 Strain and stress profile for FRP strengthened steel section under elastic deformation

Figure 5.4 Strain and stress profile for FRP strengthened steel section with both compression yielding

Figure 5.5 Strain and stress profile for FRP strengthened steel section with both tension and compression yielding

Figure 5.6 Resultant force components for FRP strengthened steel beam

Figure 5.7 Comparison between experimental and analytical load-deflection relationships

Figure 5.8 Comparison between experimental and analytical deflections of specimen A6

Figure 5.9 Stress-strain relationships of materials used in FE analysis

Figure 5.10 Mesh and loading and boundary conditions used in FE analysis

Figure 5.11 Convergence of finite element analysis

Figure 5.12 Comparison between experimental and numerical load-deflection relationships

(a)

(b)

Figure 5.13 Comparison of deformed shapes between experimental and FE results

(a) experimental result (b) FE result

(a)

(b)

Figure 5.14 Stress distribution in (a) steel beam and (b) FRP laminate

Figure 5.15 Comparison of experimental and numerical strain distributions along the FRP laminate of specimen A6

Figure 5.16 Comparison of experimental and numerical stress development in FRP laminate and steel bottom flange at mid-span of specimen A6

Figure 5.17 Relationship between number of FRP layers and increase of flexural capacity

Figure 5.18 Relationship between number of FRP layers and maximum FRP strain

Figure 5.19 Relationship between tensile modulus of FRP and increase of flexural capacity

Figure 5.20 Relationship between tensile modulus of FRP and maximum FRP strain

Figure 5.21 Relationship between bond length and increase of flexural capacity from numerical analysis

Figure 5.22 Relationship between bond length and maximum FRP strain

CHAPTER 6

BOND FAILURE MODEL

6.1 INTRODUCTION

An important possible failure mode for FRP bonded steel beams is debonding, i.e., separation of adhesively bonded composite laminates from the steel surface. Such failure modes, unless adequately considered in the design process, may significantly reduce the effectiveness of the strengthening scheme. Due to the potentially catastrophic nature of the debonding failure in conjunction with considerable uncertainty in determining necessary bond stresses, debonding must be handled carefully for design. The assumption of perfect bond in considering the flexural behavior must be later justified in the debonding analysis to ensure premature debonding does not occur. In comparison with the design of flexural capacity, the prediction of debonding strength of FRP strengthened steel beams is much more complicated. The flexural behavior is reliant primarily on the stiffness and the ultimate strain of the FRP laminates. These values may be reliably determined from standardized testing and should be guaranteed by the manufacturer of the FRP materials. Unlike the design of flexural capacities in which the theoretical prediction can be easily performed based on the material properties, the prediction of debonding failure of FRP strengthened steel beams is much more complicated because besides material properties the bond strength is highly dependent on the geometries, loading conditions, processing techniques and terminal details of the bond.

In recent years, some efforts have been made on this important issue through both experimental and theoretical investigations (Matta, 2003; El Damatty and

Abushagur, 2004; Fawzia et al., 2004; Schnerch et al., 2005; Hollaway et al., 2006). Some models on bond failure prediction of FRP bonded steel beams have been proposed based on principle stress theory (Cadei et al., 2004; Schnerch et al., 2006), singularity theory (Lenwari et al., 2006) and fracture mechanics (Colombi, 2006; Greco et al., 2007). However, the applicability and effectiveness of these models are more or less limited due to the hypotheses made to simplify the problems coming from material nonlinearity, complex geometry at the end of bondline, stress concentration and singularity.

The main aim of this chapter is to derive and assess a model for the prediction of possible bond failure for FRP strengthened steel beams. To this purpose, the mechanism of bond failure was discussed based on the experimental observations. With the submodeling technique used in FE analysis, the detailed stress/strain distribution along the bondline was checked out. A strain energy based bond failure model was proposed based on the experimental results. The validity of the model was then assessed by comparing the predicted debonding strength with the experimental results on FRP strengthened beams with bond failure. Furthermore, the proposed bond failure model was incorporated into the FE simulation. The flexural capacity decrease caused by bond failure was successfully realized in the FE modeling. The effects of different parameters including thickness of FRP laminate, tensile modulus of laminate, thickness of epoxy adhesive and bond length on debonding failure were explored based on the results from subsequent numerical analyses.

6.2 BOND FAILURE MODEL

With the experimental results of FRP strengthened steel beams with debonding failures as well as FRP bonded steel joints discussed in the preceding chapters, FE

models of strengthened beams and joints with different geometries and strengthening parameters were created. To improve the efficiency of the FE analysis, submodeling technique was used in the modeling. An equivalent stiffness method was proposed and applied to the FE model. The stress/strain distribution and development along the bondline between FRP and steel especially at the terminal of the laminate were investigated in detail. Based on the analyses of both numerical and experimental results, a bond failure model for prediction of the debonding of FRP laminate was proposed.

6.2.1 Modeling Strategy

In FRP bonded steel beams, the highly concentrated stress is generally distributed in the region at the end of the bondline. To investigate the stress and strain distribution at this region, a special modeling technique, i.e. the so-called submodeling technique, was used in the FE modeling. The idea of submodeling as applied to the problem of FRP strengthened steel members is as follows. Only those regions where the stress/strain is highly concentrated due to material discontinuity should be analyzed in a great detail, i.e. using extremely fine nonuniform element meshes, which are close to the ends of the FRP laminates. Accordingly, the displacement and stress fields will be calculated in several steps, by performing FE analysis for a successively reduced local region. In this multi-step procedure, the nodal displacements obtained in some predecessor steps will be applied as the external geometric boundary conditions when performing the corresponding successor step.

Another important feature of the submodeling approach is that after the initial, global analysis has been performed, the chosen local regions are fully separated and further analyzed independently of each other. Instead of performing global stress analysis for the whole structure with progressively refined element mesh, one or

more significantly smaller parts of the structure are analyzed in each consecutive step. Figure 6.1 shows the mesh of global model and local submodel for the steel beam strengthened with FRP laminate used in current FE analysis. Only a small zone around the end of the bondline was abstracted from the global and meshed with highly densified elements in the submodel. The mesh had the maximum density at the end of the adhesive layer, where the degree of stress concentration was most serious.

6.2.2 Equivalent Stiffness Method

For composite structures, solid element is sometimes preferred because the interfacial shear stress is always of concern in these cases while through-thickness shear can not be considered in shell elements. To investigate the stress distribution in bondline of the FRP-strengthened beams, Deng et al. (2004) used 4-node doubly curved shell elements to model steel beam while 8-node linear solid elements were employed for the adhesive and the carbon laminates in ABAQUS.

The thickness of adhesive layer was much smaller than the dimensions of joint length and width. Therefore, the number of elements could be very large if solid elements were used, and the numerical analysis would be almost impossible to perform due to the unimaginable computing time. Due to this problem, in the numerical modeling plane strain elements were often employed to model the plate composite structures since in these structures the plate length and width are very large compared to the dimension in thickness direction (Sheppard et al., 1998; Oh et al., 2003; Du et al., 2004).

Considering the validity and practicability of numerical modeling with exact adhesive layer between FRP and steel, the plain strain elements were believed the

best choice in this study. However, there was a remarkable difference in the FRP-retrofitted steel I-beam compared to the composite plate structures mentioned above that the width of the upper adherend was not uniform through the thickness. To solve this geometry problem, an equivalent stiffness method was used to make the plain strain elements applicable in the modeling.

For an I-beam with uniform material, the bending resistance stiffness is expressed as EI , where E is the Young's Modulus of the steel, I is the moment of inertia of the beam's cross section. By separating the contribution of web and flange, this stiffness can be expressed as

$$EI = EI_w + EI_f \quad (6.1)$$

where I_w and I_f are the area moment of inertia of the web and flange respectively with respect to the neutral axis of beam. For the beam with equal upper and lower flange section area, the first component is defined as follow:

$$I_w = t_w(d - 2t_f)^3 / 12 \quad (6.2)$$

where t_w is the web thickness, d is the beam depth, t_f is the flange thickness, I_w and I_f are the area moment of inertia of web and flange with respect to the neutral axis of beam respectively. The cross-section configuration is shown in Figure 6.2(a). Assume the web be substituted by another web with the same depth as the existing one, as shown in Figure 6.2(b), but with a larger width b which equals that of the flange. Consequently, the relevant area moment of inertia of this web is

$$I'_w = b(d - 2t_f)^3 / 12 \quad (6.3)$$

If the former web is substituted by the wider one, to keep the stiffness unchanged, namely,

$$EI_w = E'I'_w \quad (6.4)$$

then the Young's modulus of the later web should be,

$$E' = EI_w / I'_w = Et_w / b \quad (6.5)$$

Therefore, the existing web can be substituted by another which has a same width with the flange without change in the bending stiffness. Since strength of the beam is dominated by the bending stiffness, this substitution is quite reasonable because beams with restrained boundary conditions are prevented from buckling before failure. In fact, in typical bridges and other structures constructed using steel I-beams, the beams generally have the compression flange supported by the top slab or lateral restrained members. It means that this method can be used quite widely to model practical beam structures.

In the numerical modeling, post-elastic behavior is involved by employing bilinear material constitutive model. Similar to the equivalent stiffness transformation, yield stress and ultimate stress of the steel web should be substituted by the respective equivalent stress so as to keep the global stiffness of the beam unchanged. A necessary step for this objective is to make the yield strain of the transformed web equal to that of the former one, i.e.

$$\varepsilon'_y = \varepsilon_y \quad (6.6)$$

Substituting the strains by corresponding stresses and modules, this relationship can be expressed as,

$$\sigma'_y E' = \sigma_y E \quad (6.7)$$

From Equation 4.5, E' can be substituted by Et_w/b in above equation. The equivalent yield stress of the transformed web is expressed as,

$$\sigma'_y = \sigma_y t_w / b \quad (6.8)$$

Similarly, the equivalent ultimate stress can be deduced in the following form,

$$\sigma'_u = \sigma_u t_w / b \quad (6.9)$$

With the substitution of the web, the FRP-bonded I-beam can be regarded as a composite sandwich beam with uniform width. Therefore, in the numerical

modeling, the plane strain elements can be used for both adhesive and adherends as mentioned earlier.

6.2.3 Stress/Strain Distribution

To have a fundamental understanding of the bond failure mechanism for FRP strengthened steel beams, the detailed stress analyses especially for adhesive layer were performed by means of numerical simulation of the tested specimens. Figure 6.3 shows the distributions of shear and normal stresses along the bondline of the FRP strengthened steel beams. It can be seen that both the maximum shear and normal stresses appeared at the end of the bondline. The maximum value of normal stress was much lower than that of the shear stress. The variations of both stresses were very remarkable at the end of the bondline. The development length was less than 100 mm. In the bondline far from the end, i.e. a location with distance to the bondline end more than 100 mm, the stress magnitude was very low and there was no more variation of shear or normal stress.

The variations of stress level at the end of bondline for specimens with different geometries are illustrated in Figure 6.4. The stress contours for all the specimens were taken at the load level of 50 kNm. Under this load level the steel beam has yielded which is corresponding to the experimental result. Since the variation of stress level is more of interest rather than the stress magnitude, to facilitate the illustration of stress variation in the stress highly concentrated area, instead of using specific stress values, the gradual variation of stress level is indicated by the numbered contour lines, i.e. the smaller the number on the contour line is, the larger the magnitude of the stress. It can be seen that the stress distribution at the end of bondline was highly dependent of the local geometry. As expected, for the specimen with no fillet or taper at the end of bondline, the stress concentration

appeared at the interfacial corner between steel and epoxy. For specimens with fillet and/or taper, the maximum shear stress was close to the corner of the FRP laminate where stress concentration was caused by the material discontinuity. For specimen with reverse taper, there was no stress concentration around the tip of the laminate, which explained why the specimen with reverse taper at the end of bondline had the maximum bond failure load.

6.2.4 Mechanism of Bond Failure

The criteria used to assess the nature and magnitude of the stresses and strains within the adhesive joint include maximum values of stress, strain or energy. In FE modeling, at the corners of bi-material interfaces where the adherend or adhesive terminates, singularity, i.e. a point of theoretically infinite stress, always exists. Clearly, the maximum value failure criteria become meaningless as these values are a function of how well the singular field is modeled, i.e. the refinement of the mesh. However it has been stated by Adams and Harris (1987) and Zhao (1991) that sharp corners do not exist in real joints and therefore neither do the stress/strain singularities. This is a valid point but as observed by and Adams et al. (1997) the value of the maximum stress/strain components is highly sensitive to small changes in the degree of rounding whereas the experimental results are not. It can be concluded therefore that maximum stress/strain failure criterion is not suitable for adhesive joints unless some averaged value is used.

Unlike peak value criteria which are strongly dependent on the local stresses, the application of a stress or strain at a distance criterion is capable of predicting insensitivity to local geometry, but only by ignoring the stress field locally and applying the criterion far enough away from the discontinuity to be clear of its influence. High stresses and strains locally to the corners of the adhesive joint are

often predicted by the modeling, even at very low loads where local cracking should not have taken place. These local strains are generally in excess of what is normally required to fail a bulk adhesive specimen and hence it is difficult to explain the presence of these strains without crack formation and propagation. However, scale effect can explain these strains, as will be shown later, by consideration of the volume over which they act as well as their magnitude.

With the scale effect taken into account, weakest link theory is based on the assumption that the material is made up of smaller elements linked together and that failure of the material as a whole occurs when any one of these elements or links fail (Sutherland et al., 1999). The probability of failure of each link subjected to a stress increase from 0 to σ is described by the distribution function $f(\sigma)$. The probability of survival of that link is then given by:

$$S(\sigma) = 1 - f(\sigma) \quad (6.10)$$

It is also assumed that $f(\sigma)$ describes the strength distribution for every element and that each $f(\sigma)$ is an independent randomly distributed variable. The probability of survival of n elements in series is then given by:

$$S(\sigma) = [1 - f(\sigma)]^n. \quad (6.11)$$

Hence the probability of failure of a chain of n elements is given by:

$$F_n(\sigma) = 1 - [1 - f(\sigma)]^n \quad (6.12)$$

The function $f(\sigma)$ may be described generally as:

$$f(\sigma) = 1 - \exp[-\varphi(\sigma)] \quad (6.13)$$

This equation forms the basis of statistical weakest link theory. A specific form of $\varphi(\sigma)$ was put forward by Weibull (1951) and is still used widely today. This function is given by:

$$\varphi(\sigma) = \left(\frac{\sigma - \sigma_{th}}{\sigma_0} \right)^m \quad (6.14)$$

Where σ_{th} is the threshold stress below which failure does not occur and σ_0 and m are called the scale parameter and the shape parameter, respectively. This form is termed the three parameter distribution. For the strength of a material subjected to an increasing load from zero to σ there is zero chance of failure only when there is no applied load. Hence, σ_u is usually taken to be zero and the two parameter form is used. This leads to a probability of failure of n elements in series of:

$$F_n(\sigma) = 1 - \exp\left[-n\left(\frac{\sigma}{\sigma_0}\right)^m\right] \quad (6.15)$$

The shape parameter m is corresponding to the shape of the distribution or the dispersion of the data. The calculation of its value is introduced in the next section. Considering a volume of material comprising of small elemental volumes, dv , instead of a chain of links gives:

$$F_n(\sigma) = 1 - \exp\left[-\int\left(\frac{\sigma}{\sigma_0}\right)^m dv\right] \quad (6.16)$$

An equivalent constant interfacial stress σ_e , is then calculated. This is defined as the constant stress which when acting on the same volume of material as the actual variable stress distribution, gives equal probability of failure:

$$F_n(\sigma) = 1 - \exp\left[-v\left(\frac{\sigma_e}{\sigma_0}\right)^m\right] \quad (6.17)$$

Equating these two equations above gives:

$$\sigma_e = \left(\frac{1}{v} \int \sigma^m dv\right)^{1/m} \quad (6.18)$$

6.2.5 Strain Energy Density Model

For adhesively bonded joint, the development of the debonding can be deemed as a crack propagating along a predefined path. In the fracture mechanism theory, a crack is initiated when the interfacial traction attains the maximum interfacial

strength and the crack advances when the work of fracture equals to the material's resistance to crack propagation. Under mixed mode loading the crack initiates when the mixed mode fracture criterion is satisfied. The mixed mode fracture criterion was given by (Tong 1998, Yang and Thouless 2001, Goyal and Klug 2004):

$$\left(\frac{G_I}{G_{IC}} \right)^{\lambda_I} + \left(\frac{G_{II}}{G_{IIC}} \right)^{\lambda_{II}} = 1 \quad (6.19)$$

where G_I and G_{II} are energy release rates under mode one and mode two loading, respectively. G_{IC} and G_{IIC} are critical energy release rates under mode one and mode two loading condition. λ_I and λ_{II} are material parameters that determine the failure locus. It was found by Goyal and Klug (2004) that with $\lambda_I = \lambda_{II} = 1$, the predicted bond failure strength agreed well with experimental results.

According to Tong and Steven (1999), the mixed-mode failure criterion for solid fracture can be expressed in a linear equation with respect to energy release rates, which is a quadratic variable with respect to stress or strain. A quadratic variable about strain, strain energy density was found to be a promising index for prediction of bond failure between composite and metal structures (Hart-Smith 1988). The general expression of total strain energy density is,

$$w = \sum_{i,j=1}^3 \int_0^{\varepsilon_{ij}} \sigma_{ij} d\varepsilon_{ij} \quad (6.20)$$

where σ_{ij} and ε_{ij} are stress and strain tensors, respectively. It is known, in the realm of continuum mechanics, that mechanical work supplied by external forces is stored in the material as two separate components of strain energy, namely to change the volume and to change the shape of the specimen. Therefore, the total strain energy density can be expressed as follows:

$$w = w_v + w_d \quad (6.21)$$

Where w_v and w_d are the dilatational and distortional strain energy density, respectively.

According to the theory of plastic flow, the unit volume change θ , is an strain proportional to the hydrostatic pressure, p :

$$\theta = \frac{1}{K} p \quad (6.22)$$

where $\theta = \sum_{i=1}^3 \varepsilon_i$, $p = \frac{1}{3} \sum_{i=1}^3 \sigma_i$, ε_i and σ_i are the principal strains and stresses and K is the bulk modulus of the material.

The dilatational strain energy density increment is:

$$dw_v = p d\theta \quad (6.23)$$

So, the total amount of dilatational strain energy density stored in the material is:

$$w_v = \int p d\theta = \frac{1}{2K} p^2 = \frac{1}{18K} \left(\sum_{i=1}^3 \sigma_i \right)^2 \quad (6.24)$$

From equations (6.21) and (6.24), the total amount of distortional strain energy density can be derived as:

$$w_d = \sum_{i,j=1}^3 \int \sigma_{ij} d\varepsilon_{ij} - \frac{1}{18K} \left(\sum_{i=1}^3 \sigma_i \right)^2 \quad (6.25)$$

Similar to the mix-mode criterion in fracture mechanics, the adhesive bond failure can be predicted by using the failure envelop which is composed of these two strain energy density components.

$$\left(\frac{w_v}{w_{vc}}\right)^{\lambda_v} + \left(\frac{w_d}{w_{dc}}\right)^{\lambda_d} = 1 \quad (6.26)$$

Where w_{vc} and w_{dc} are the critical dilatational and distortional energy densities, respectively. λ_v and λ_d are correlative parameters. As discussed earlier, the interaction relationship between the critical values of the two variables can be expressed linearly based on the experimental observations and numerical investigations (Fan et al., 2005). The interaction failure criterion was then defined for a unit value of the dilatational and distortional interaction function $w_r(w_v, w_d)$, written as follows:

$$w_r(w_v, w_d) = \frac{w_v}{w_{vc}} + \frac{w_d}{w_{dc}} \quad (6.27)$$

Based on the results of the numerical analyses on different types of joints, the strain energy densities at failure can be found out. The relationship between dilatational and distortional strain energy densities can be fitted into a straight line as shown in Figure 6.5. By the linear interpolation of the data, the critical values of effective dilatational and distortional strain energy densities can be found out. For joints made of the materials used in this study, the critical values for dilatational and distortional strain energy densities are found out to be 0.69 N/mm² and 3.62 N/mm², respectively.

Taking advantage of the fact that any stress operator can be used instead of the stress σ in the weakest link theory, the probability of failure of the FRP bonded steel joint is given by:

$$F_n(w_r) = 1 - \exp\left[-\int \left(\frac{w_r}{w_{r0}}\right)^m dv\right] \quad (6.28)$$

Where w_{r0} is the critical representative strain energy density, which can be determined from experimental results. Similarly, the equivalent strain energy density is given by:

$$w_e = \left(\frac{1}{v} \sum_{i=1}^{n_e} \left(\frac{w_r}{w_{r0}} \right)_i^m v_i \right)^{1/m} \quad (6.29)$$

where n_e is the number of adhesive elements in the numerical analysis and v_i is the volume of the corresponding element. Based on the numerical analysis with utilization of user-subroutine to calculate the value of the variables at each increment, the value of equivalent strain energy density at certain static load can be found out, thus the prediction of bond failure load.

6.3 EVALUATION

6.3.1 Procedure of Debonding Prediction

To apply the bond failure model to prediction of the FRP debonding, the shape parameter m must be found out first. According to Hitchon and Philips (1978), this parameter can be evaluated by the following equation:

$$m = \frac{1.2}{CV} \quad (5.30)$$

Based on the test results of FRP-steel joints presented in chapter 3, the value of m was estimated to be 11.4.

The calculation of effective strain energy density in the numerical analysis was realized by incorporating a user-subroutine into the FE model. To have an intuitional understanding of the distribution of strain energy density in the FRP strengthened steel beam, the relationship between strain energy density and the distance to the end of bondline is illustrated in Figure 6.6. It can be seen that the strain energy density was highly concentrated at the end of the bondline. Similar to the stress variation, the decrease of strain energy density from the end of bondline was very remarkable with a small range, which was around 60 mm. Beyond that

range, the magnitude of the strain energy density was very small and there was no more variation along the bondline.

The comparison of detailed distributions of the strain energy densities at the end of bondline in different types of the joints is illustrated in Figure 6.7. It can be noted that both dilatational and distortional strain energy densities were highly concentrated in the end of the joint where the discontinuity was caused by the termination of the adherend.

6.3.2 Comparison with Maximum Stress Criterion

As mentioned earlier, one advantage of this bond failure model compared to the maximum value criteria is the low sensitivity of the failure index to the variation of local geometry and number of elements used in the FE analysis. To verify this, numerical analyses on FE models with different rounding radii and various numbers of elements were conducted. The relationships of failure parameters, i.e. stress and strain density, and rounding radius are shown in Figure 6.8. It can be seen that with the radius of the corner rounding increasing, the failure stress and strain energy density used in maximum criterion decreased remarkably. On the contrary, the influence of the rounding degree on the failure parameters used in current failure model was quite limited. The relationships of failure parameters, i.e. stress and strain density, and number of elements used in the FE analyses are shown in Figure 6.9. Similarly to the effect of rounding degree, the density of elements had a significant impact on the failure parameters used in maximum value criterion, but much less influence on those used in current failure model. From these comparisons it can be seen that the accuracy of the early present numerical analysis of FRP-steel joints is dependent on the mesh of the bondline. For illustration of the stress distribution along the bondline, the adhesive is deemed as a very thin layer and the

variation of stress through the adhesive thickness is neglected, so that the modeling is accurate enough. However, for the analysis of bond failure mechanism much detailed stress variation and concentration need to be investigated. Therefore, only the advanced numerical analysis as presented in this section is accurate enough to obtain the desired result.

6.3.3 Comparison with Experimental Results

With the proposed bond failure model, the bond failure load of a FRP strengthened steel beam under flexural loading can be numerically predicted. The comparison between the predicted and the experimental bond failure loads/deflections are listed in Table 6.1. The relationship between experimental results and numerical prediction of mid-span deflections at the bond failure of different specimens is illustrated in Figure 6.10. It can be noted that the predicted results of bond failure agreed well with the experimental results.

To predict the relationship between load and deflection of FRP strengthened steel beams with bond failure, a user subroutine was incorporated into the FE analysis. The first step of the numerical analysis process was to perform FE analysis with the global model. The initial displacement u_0 and the critical displacement u_c were predefined parameters. The output variables u_{sc} and F_{sc} were used as boundary conditions for the subsequent submodel analysis. In the submodel, the user subroutine will be activated after each increment finished. The output stress and strain variables in each submodel increment were used to calculate the effective strain energy density in the user subroutine. If the critical value of the effective strain energy density was not reached, the program would continue running the next step. If the critical value of the effective strain energy density was met, the submodel program would stop running and the peak load on the beam could be

found out. The failed elements would be deactivated to simulate the bond failure between the FRP laminate and the steel beam. The deactivation of failed elements was realized by changing the modulus of the element to a very small value that can be neglected compared to the original one. The global model after bond failure would keep running with the FRP laminate separated from the steel beam. The program would stop running when the predefined critical displacement was reached. The flow chart of this procedure is illustrated in Figure 6.11. The relationships of load deflection for numerical models and experimental results of specimen C2 are compared in Figure 6.12. It can be seen that for FE analysis without bond failure incorporated, the abrupt drop of load can not be achieved in the numerical procedure, so the curve of the FE analysis after bond failure could not be compatible with that of the experiment. However, for the FE model with debonding process incorporated, the load-deflection relationship was well simulated and the ultimate load of numerical analysis was in good agreement with the experimental result. This comparison shows that the incorporation of bond failure model in the FE analysis is very important for the real flexural behavior of FRP-strengthened steel beams to be modeled accurately.

The proposal of the bond failure model to predict the bond failure of FRP strengthened steel beam is an original work. In the model, the effective strain energy density is developed as the criterion for bond failure. Compared to the other existing stress-based failure model, the proposed model has the following advantages: i) unlike the stress-based models, the impact of adhesive thickness on the bond strength can be predicted correctly, which is discussed in detail in the next section, ii) the bond strength of bondline with different end details can be predicted, iii) the influence of mesh density on the predicted result is much less than that on the result predicted from stress-based models.

6.4 PARAMETRIC STUDY

Various parameters influence the maximum values of strain energy density in the bonding region. To understand the effects of different strengthening parameters on the bond failure load of FRP strengthened steel beam, analysis of FE models with various combinations of different parameters was conducted. The effects of the following parameters on the debonding strength were investigated: i) thickness of FRP laminate, ii) elastic modulus of the FRP laminate, iii) thickness of adhesive layer and, iv) bond length. The parametric beam used was similar to specimen C2. The material properties and loading conditions were same as those used in the experimental program. When the impact of a specific parameter was analyzed, the other parameters were kept constant.

6.4.1 Thickness of FRP laminate

The first parameter investigated was the thickness of the FRP laminate. The thickness of the laminate was varied, keeping the width of the laminate the same. Figure 6.13 shows that increasing the thickness of the FRP laminate was almost linearly proportional to the increase of the strain energy density, although it had a less significant effect when the magnitude was higher. With the thickness of the laminate varying from 1.4 mm to 12.5 mm, the strain energy density had an increase of the seventy-eight percent.

6.4.2 Tensile Modulus of FRP laminate

Another parameter of interest is the tensile modulus of FRP laminate. With the rapid progression of material technology, the modulus of composite material is believed to be able to be enhanced dramatically in the coming years. Therefore the

laminate with tensile modulus up to 1000 kN/mm^2 was employed in the numerical study. Figure 6.14 shows the effect of the laminate modulus on the strain energy density. The tensile modulus of the bonded plate varied from 150 kN/mm^2 to 1000 kN/mm^2 . Similar to the effect of laminate thickness, the strain energy density increased when the modulus of the plate increased.

6.4.3 Bond Length

The most important of the parameters studied was the influence the bond length of the FRP laminate. The increase of bond length may lead to an increase of bond failure load as a result of two mechanisms. First, for a simply supported beam the closer is the location to the end of the beam, the smaller the strain distributed in the tension flange. Therefore, the shear deformation required in the end of the bondline is smaller when the end of the laminate is closer to the end of the beam, thus a larger bond failure load. Second, the increase of the bond length can reduce the longitudinal stress distributed in the tension flange so that the stress in the adhesive can be lowered. The effect of the second mechanism is much less significant compared to that of the first. Figure 6.15 shows the relationship between bond length and normalized strain energy density. The bond length varied between 700 and 2200 mm. It can be seen that the strain energy density dropped dramatically with the increase of the bond length. The strain energy density was reduced to a very small level when the bond length approached the span of the beam. Based on this observation, it can be concluded that increase of bond length is the most effective and operable measure to enhance the bond failure resistance capability of FRP strengthened steel beams.

6.4.4 Thickness of Adhesive Layer

Figure 6.16 shows the effect of the adhesive layer thickness on the strain energy density. It can be seen that the increase of strain energy was almost linearly proportional to that of the adhesive thickness. This relationship between bond failure index and adhesive thickness agreed well with the experimental and analytical results from the research of Gleich et al. (2001) and Sawa et al. (2000). This finding explains why most adhesively bonded structures with thicker bondline result in weaker joints. However, in the prediction with the bond failure models developed by some other researchers (Lenwari et al., 2006; and Deng and Lee, 2004), the relationship between adhesive thickness and bond failure load was reversed, i.e. the bond failure load went up with the increase of the adhesive thickness, which was obviously inconsistent with the truth. Therefore, the bond failure model proposed in this study has been demonstrated to be more reliable in a sense.

6.4.5 Concluding Remarks

Based on the numerical analysis of FRP strengthened steel beams, the impact of different parameters including thickness and modulus of FRP, thickness of adhesive and bond length on the bond failure has been investigated. The conclusions are drawn as follows:

- 1) The bond failure load of FRP strengthened steel beams decreases with the increase of thickness or modulus of FRP laminate, when the other parameters are constant. The impact of these two parameters on the bond failure load is less significant than the others.

- 2) Increasing the bond length results in significant increase of bond failure load.
This is the most effective measure to enhance the bond failure resistance capacity of FRP strengthened steel beams.
- 3) The thinner the adhesive layer is, the higher the bond strength of FRP strengthened steel beams.

Table 6.1 Comparison between numerical and experimental results

Specimen	D_e (mm)	D_f (mm)	$D_f - D_e$ (mm)	$\left \frac{D_f - D_e}{D_e} \right $ (%)	Failure mode in experiment	Failure mode in numerical analysis
C2	20.5	18.1	-2.4	11.7	DY	DY
C3	18.7	15.3	-3.4	18.2	DY	DY
C4	14.6	13.8	-0.8	5.2	DY	DY
C5	25.2	26.6	1.4	5.6	DY	DY
C6	26.1	22.5	-3.6	13.8	DY	DY
C7	19.0	15.5	-3.5	18.2	DY	DY
C8	12.3	11.4	-0.9	7.1	DY	DY
C9	28.1	24.6	-3.5	12.4	DY	DY
C10	30.3	27.3	-3.0	9.8	DY	DY
C11	33.1	30.4	-2.7	8.1	DY	DY
C15	17.9	17.2	-0.7	4.1	DY	DY
C16	21.1	20.6	-0.4	1.7	DY	DY
C17	33.4	29.2	-4.2	12.7	DY	DY

Note: D_e – Experimental mid-span deflection; D_f – Numerical mid-span deflection; DY – Steel yielding following or followed by debonding of FRP laminate.

Figure 6.1 Mesh of global model and submodel of FRP strengthened steel beam

Figure 6.2 Cross-sectional view of equivalent stiffness transformation for I-beam

Figure 6.3 Stress distribution along bondline of FRP strengthened steel beam

Note: the change (increase) of the numbers from 1 to 9 indicates the variation (decrease) of the stress level from the highest to the lowest

Figure 6.4 Stress variations at the end of bondline of FRP strengthened steel beams: (a) normal stress variation without fillet; (b) shear stress variation without fillet; (c) normal stress variation with half fillet; (d) shear stress variation with half fillet; (e) normal stress variation with full fillet; (f) shear stress variation with full fillet; (g) normal stress variation with taper; (h) shear stress variation with taper; (i) normal stress variation with reverse taper; (j) shear stress variation with reverse taper

Figure 6.5 Relationship between dilatational and distortional strain energy densities

Figure 6.6 Distribution of strain energy density along the bondline of FRP strengthened steel beam

(d)

(e)

Note: the arrow indicates the gradual variation (decrease) of the strain energy density from the highest to the lowest level

Figure 6.7 Distribution of strain energy density at the end of bondline of FRP strengthened steel beams: (a) without fillet; (b) with half fillet; (c) with full fillet; (d) with taper; (e) with reverse taper

Figure 6.8 Effect of degree of rounding on effective and maximum strain energy densities

Figure 6.9 Effect of mesh density on effective and maximum strain energy densities

Figure 6.10 Comparison between experimental and numerical results

Figure 6.11 Procedure of FE modeling with debonding

Figure 6.12 Comparison between experimental and numerical load-deflection curves

Figure 6.13 Relationship between effective strain energy density and laminate thickness

Figure 6.14 Relationship between effective strain energy density and tensile modulus of laminate

Figure 6.15 Relationship between effective strain energy density and bond length

Figure 6.16 Relationship between effective strain energy density and adhesive thickness

CHAPTER 7

CYCLIC BEHAVIOR OF FRP STRENGTHENED STEEL BEAMS

7.1 INTRODUCTION

Bridges and marine structures usually have a large number of cycles of significant live load. In some cases, controlling fatigue is practically even more important than controlling static failure. A large number of existed steel members are often required to be strengthened or repaired to enhance their fatigue resistance due to some inevitable reasons such as, the need to increase the service load, corrosion, and the lack of proper maintenance. As discussed earlier, the traditional method to strengthen or to repair steel structures has several disadvantages, including the difficulty of installing heavy steel plates at the bridge site, the limited length of plate that can be delivered to the bridge site, etc. Therefore, there is a current need to develop a new rehabilitation method that can be used either together with or separately from hole drilling that does not require welding or mechanical fastening of additional steel plate.

The adhesive bonding of FRP laminates to the bottom of the tension flange of flexural member is a potential method for strengthening and repairing steel structures subjected to cyclic loading. The superior mechanical and fatigue properties of FRP materials make them excellent candidates for rehabilitation and retrofit of these structures. FRP is particularly appealing for the fatigue life extension of old steel bridge structural elements. The carbon fibers have very high fatigue resistance that prevents crack propagation from the steel section to the composite patch. FRP laminates can endure over 1 million cycles of fatigue loading with a stress range of one-half of the ultimate strength (Lorenzo and Hahn 1986).

Moreover, the high stiffness of FRP reduces the stress range in the cracked steel section and promotes crack bridging.

During the past decade, there have been many studies on repair and retrofit of concrete beams with epoxy bonded FRP materials. However, very few studies have addressed the use of epoxy bonded plates or sheets for strengthening steel beams under cyclic loading. Tavakkolizadeh and Saadatmanesh (2003c) retrofitted six artificially damaged small-scale steel beams by bonding one layer of 300 mm long FRP laminates. They found that the retrofitted specimens experienced longer fatigue lives of between 2.6 to 3.4 times the un-retrofitted specimens for stress ranges of 345 to 207 N/mm².

This chapter discusses the effectiveness of adhesive bonding of FRP laminates to steel flexural members to improve their fatigue resistance capacities. Three series of experiments were conducted. In the first series, FRP bonded steel plate joints were tested under cyclic tensile loading. The relationship between the fatigue life and the stress level was investigated. Second, full-scale test on FRP strengthened steel beams under cyclic loading was carried out to study the performance of bond between FRP laminate and steel substrate. The effect of fatigue life extension by bonding FRP laminates to damaged steel beams was investigated in the final experimental program.

7.2 FRP BONDED STEEL JOINTS UNDER CYLCIC LOADING

The use of adhesively bonded joints subjected to cyclic loading has increased significantly during the last two decades, not only in the aerospace industry but also in other fields such as marine and construction. The superior fatigue resistance of adhesive joints compared to riveted joints has been reported as far back as more

than twenty years ago (Adams and Wake, 1984). The fundamental concept for fatigue design is based on plotting the stress amplitude versus the fatigue life in cycles (S–N curve) using experimental data. For some metals such as steel, there exists a critical stress value below which the material can undergo an apparently unlimited number of cycles without fatigue failure. This stress value is usually referred as the endurance limit or fatigue limit. In an adhesively bonded joint, the presence of an endurance limit indicates that a threshold stress value exists below which cracks will not initiate or propagate in the adhesive layer. This section presents the investigation of the fatigue behavior of FRP adhesively bonded steel joints under different levels of load amplitudes. The study was carried out by experimental investigation on double-lap symmetric specimens. The main objective of this experimental program was to study the fatigue behavior of the FRP-steel joints and the relationship between fatigue life and load amplitude.

7.2.1 Experimental Program

The experimental studies were carried out at the Construction Technology Laboratory, Nanyang Technological University of Singapore. In this experimental program, totally 30 specimens were tested in order to evaluate the effect of the load level on the fatigue life of FRP bonded steel plate joints. The specimen details of the FRP bonded steel joints under cyclic loading are presented in Table 7.1.

The steel plates and the FRP laminates used in the experiments had same material properties as those reported in previous chapters. The specimen configuration and test setup are illustrated in Figure 7.1. The total length of the specimen was 332.5 mm. The geometry and material property of the joint were symmetrical about the centerline of joints. The bond length l_0 was 75 mm at each side of the specimen.

The CFRP laminate and the steel plate had the same width b equal to 50 mm. The thickness of internal steel plate was 5 mm.

The steel plates were ground to expose a fresh chemically active surface to ensure a better mechanical interlocking. One layer of FRP laminate was applied on the ready-to-bond surface after grease and dust on the steel and the laminate surfaces were thoroughly removed by using methyl ethyl ketone. Other preparing procedure and specimen details are same as those employed in the static experimental program.

According to the different types of FRP laminates used, the specimens were divided into two groups: group A and group B. FRP laminate UC was used in group A and laminate SCH41 was used in group B. First, five reference specimens in each group were investigated by means of static failure experiments without precedent fatigue loading. The specimens were loaded in uni-axial tension up to failure, displacement-controlled at a rate of 0.5 mm/min. Subsequently, ten fatigue experiments were conducted in each group. The specimens were sinusoidally loaded under tension at an amplitude ratio of 0.1 and at different upper boundary values from 30% up to 70% of the average static failure load of the reference specimen. The comparison of strain variations along bondline at different load levels is shown in Figure 7.2. As expected the stress concentration level at the end of the bondline increased with the external load increase. Two specimens were tested at each level of same load amplitude. One was instrumented on the outside of the laminate by two strain gauges. The gauges were placed in center of the laminate in order to measure the axial strain progression in longitudinal direction during the fatigue experiments. The fatigue experiments were performed load-controlled at frequency of 20 Hz. No increase in temperature was observed at this frequency. The

fatigue experiments were conducted up to bond failure between FRP laminate and steel plate or to a maximum of 10 million load cycles, whichever was earlier.

7.2.2 Failure Modes

All the tested specimens showed an adhesive failure mode. The rupture happened at the adhesive–steel interface, since the bond between FRP laminate and epoxy is stronger. Two types of rupture paths were observed during the test as shown in Figure 7.3. They were named as Z and U ruptures according to the shapes of the rupture paths, respectively. The failure was caused by the high stress concentration at the end of the bondline. Both types of failure modes happened at almost same probability due to the random bond properties at both side of the joint.

The bond failure for most specimens was very brittle without announcement by visible cracks on the specimen surfaces or deformation. The progression of the average strain amplitude, measured from the strain gauges on the FRP laminate, is shown in Figure 7.4 for some selected experiments. The abscissa has been normalized with respect to the number of cycles at failure N_f . The strains remained almost constant and, therefore, showed no obvious sign of damage initiation or progression in the FRP-steel joint.

7.2.3 Fatigue Life

The applied load versus the logarithm of number of cycles to failure for specimens in group A and group B are presented in Figure 7.5 and Figure 7.6, respectively. It can be noted that the relationship between the logarithm of the number of cycles to failure N_f and the normalized stress is more or less linear. The best-fit curves were produced linear regression analysis produced. It can be seen that the difference

between the equations of the two curves is very small. Specimens tested under 30% of the maximum load did not fail even after ten million cycles. Therefore, the fatigue threshold limit for the two groups of specimens is taken as 30% of the ultimate failure stress under static loading.

7.2.4 Stiffness Variation

For most specimens, no obvious variation of stiffness was observed in the experimental program. For specimens tested at load amplitude of forty percent of the ultimate static strength, a degradation of stiffness was noticed just before the final bond failure was reached. Figure 7.7 shows the first and the last load-elongation cycle of specimen A12 under cyclic amplitude equal to 40% of the ultimate bond strength. A considerable increase of the elongation and a corresponding loss in stiffness during the fatigue experiment can be seen. For the first cycle, the hysteresis loop is relatively small compared to that of the last cycle.

7.2.5 Effect of Fatigue on Bond Strength

The load-deflection curves of two specimens under static tensile loading are plotted in Figure 7.8. These two specimens had no difference except the testing procedure. The first was directly tested to failure under monotonic tension. The second was axially tensioned to failure after ten million cycles fatigue test at 30% of tensile strength of the joint. Both of these two specimens failed in a brittle manner. After ten million fatigue cycles applied to the second specimen, there was no crack or damage detected by visual inspection. It can be seen that the reduction in bond strength was not significant. The residual tensile strength after cyclic loading was 91% of the initial tensile load. This phenomenon was similar with that observed by Liu et al. (2005). Therefore it may be concluded that the degradation of bonding

strength did not occur at 30% fatigue load level of static bonding strength after ten million fatigue cycles.

7.3 BOND PERFORMANCE OF FRP STRENGTHENED STEEL BEAMS UNDER CYCLIC LOADING

Up to date, the research on bond performance of FRP strengthened steel flexural members under cyclic loading is very limited. Although the cyclic behavior of FRP bonded steel joint has been presented in the foregoing section, no qualitative relationship has been established between the fatigue life and external load for FRP bonded flexural members under cyclic loading. The principal aim of the work presented in this section was to investigate the fatigue performance of the adhesive bond between the FRP laminate and the steel substrate, particularly in the regions where the FRP laminates terminated. This was achieved through a fatigue test program of FRP laminate strengthened steel beams subjected to mechanical cyclic loading with various load levels.

7.3.1 Experimental Program

Fourteen steel beams strengthened with FRP laminates were tested. The steel beams had same material property, sectional dimension and surface preparation as those used in the last group in the static experimental program. The specimens were divided into two groups according to different FRP materials used for the strengthening. Laminates UC and SCH41 were employed in group A and group B respectively. One layer of laminate was used for each specimen. The length of each beam was 2400 mm in group A and 1800 mm in group B. The bond lengths for specimens in two groups were 750 mm and 700 mm respectively. The details of the specimens are summarized in Table 7.2. The first specimen in each group was used

as control specimen and tested under monotonic loading until debonding of FRP laminate.

The fatigue tests were carried out in a hydraulic test machine with a maximum capacity of 1000 kN, using a four-point bending setup, as shown in Figure 7.9. A spreader beam was installed below the swivel to distribute the concentrated load. Deflections were measured at three locations using LVDT. To investigate the phenomenon of crack initiation and growth, three 2 mm long strain gauges were mounted on the face of the FRP laminate along the longitudinal centre line at both ends and center of the laminate. All data, including the load values measured by the sensor in the hydraulic system, were automatically recorded by a data logging system.

All the specimens were subjected to uniform cyclic loading applied by four-point bending. A minimum load of 5 kN was applied to all the fatigue tested beams so as to ensure firm contact between the beam and the supports. According to the research of Emberson and Mays (1996), for fatigue test at frequencies much in excess of 2 Hz the beam was unable to recover fully from one load application before the arrival of the next. A high frequency may also produce undesirable heat effect on the beam. On the other hand, the test could be very time-consuming if the frequency is too low. Therefore, the frequencies of 1 Hz and 2 Hz were used for specimens tested under load levels of more than and less than 60% ultimate load respectively.

7.3.2 Stress/Strain Variation

The strain development at different locations measured in the specimen SA5 during the fatigue tests are shown in Figure 7.10. It can be observed that the strains at all

locations remained constant during the first thirty percent of fatigue cycles. After that the strain on the right began to drop gradually. Later on the strain on the left began to decrease as well. Therefore, bond failure on the left happened later than that on the right. The strain decrease at the centre initiated after sixty percent of total cycles. When two thirds of the fatigue life of the specimen was reached, the strain at each end approached zero indicating the bond at the end was completely invalid. The strain in the center of the laminate dropped rapidly from a constant maximum value to a much lower value close to zero, which indicated the bond between the FRP laminate and the steel beam was almost completely damaged by the fatigue loading.

7.3.3 Stiffness Variation

For steel beams bonded with FRP laminates subjected to cyclic loading, the decrease of flexural stiffness could be caused by the gradual degradation of the adhesive bond between the strengthening laminate and the substrate. In the current study, the variation of flexural stiffness of the specimens was observed by the change of the deflection at mid-span of the beam. Figure 7.11 shows the deflections measured from the three specimens during the fatigue test. It can be observed that before crack initiating the stiffness of the beam remained constant. With crack growth, the deflections obtained increased in gradual manner, until the crack cross the middle section of the retrofitted beam. The increase of deflection was small because of the thinness of the plate compared to that of steel beam. Similar observations were made on the other specimens. For the specimen tested under lower load amplitude, the degradation of stiffness happened relatively later than those subjected to higher load amplitude.

7.3.4 Failure Modes

The crack initiation and propagation in all the specimens were similar except for specimen SB2. The load amplitude on this specimen was too high and caused a sudden bond failure at one end of the strengthening laminate after only 25 cycles. Figure 7.12 shows the crack development plot for a selected specimen, which was typical for all the other specimens. It can be seen that, the initiations of the cracks at left and right ends were not simultaneous. Once initiated, the crack at each end grew rapidly, but then almost stopped after one of the two cracks had advanced to certain extent. The typical failure mode of the specimens is shown in Figure 7.13. For all the specimens, the crack initiated from one end of the FRP laminate. With fatigue cycles increasing, the crack propagated along the interface between the steel beam and the adhesive and moved towards the centre of the bondline. Since there was always residual bond remained and FRP laminate and steel beam can not be completely separated, the crack stopped propagating eventually.

7.3.5 Fatigue Life of Bond

The relationships between the normalized load and the logarithm of number of cycles to failure of specimens in the two groups are presented in Figure 7.14 and Figure 7.15 respectively. The relationship in each group is more or less linear. Linear regression analysis produced the following best-fit curve for specimens in group A:

$$P_N = -9.97 \text{ Log } (N_f) + 101.47 \quad (8.1)$$

where P_N is the normalized load amplitude in the fatigue test, N_f is the number of cycles to failure. Similarly, the linear relationship for specimens in group B is as follows:

$$P_N = -10.12 \text{ Log } (N_f) + 98.55 \quad (8.2)$$

The last specimen in each group did not fail even after three million cycles. Furthermore, the variation of the strain at the end of laminates was still very small after three million cycles, which indicated that the specimens were not likely to fail in fatigue under such a load level. Therefore, the fatigue threshold for the tested beams is regarded as thirty percent of the ultimate failure load that will cause debonding of FRP laminate under static loading.

7.4 FATIGUE STRENGTH OF FRP REPAIRED STEEL BEAMS

FRP laminates can be adhesively bonded to fatigue sensitive details in steel members to enhance their strength and fatigue life. During the past decade, there have been many studies on repair of fatigued concrete beams with epoxy bonded FRP materials. However, very few studies have addressed the use of adhesively bonded laminates to repair damaged steel beams subject to cyclic loading. This section describes the full-scale experimental investigation of FRP retrofitted steel beams under repeated cyclic loading. For each specimen, four-point bending was applied with constant amplitude and frequency until failure occurred. The fatigue lives of the beams with and without FRP materials were compared. Different failure modes were observed and discussed. The mechanism of fatigue life extension with FRP laminates was analyzed. It was found that the fatigue life can be enhanced by more than three times by using these laminates.

7.4.1 Experimental Program

In the experiment program, wide flange steel I-beams were employed for specimen fabrication. The steel beams used in the experiments had same sectional properties as those used in the second group of static beam experiments. The details of

material properties, sectional geometries and preparation procedure of the specimens have been presented in the chapter 4 and are not repeated here.

To simulate the serious loss of flexural capacities due to long term deterioration, the steel beams were artificially damaged before application of external bonded FRP laminates. Two types of artificial damages were created on the tension flange of the specimens. For the specimens with first type of damage, both edges of the tension flange at mid-span were created two through cuts in the tension flange of each specimen as shown in Figure 7.16(a). In the second type, a notch with 2.5 mm depth and 20 mm width was produced at mid-span of each specimen as shown in Figure 7.16(b). The length of the cut was 20 mm and the width was 6 mm. The first specimen in each group was designed as control specimen, hence no FRP laminate bonded. The other specimens were externally bonded with FRP laminates with different configurations. The details of the configurations of the specimens are summarized in Table 7.3. All of the specimens were subjected to uniform cyclic loading applied by four-point bending. The applied load had a frequency of 1 Hz and a load ratio of 0.1. The maximum load was 210 kN, which was about fifty-five percent of the ultimate strength of the intact steel beam.

Based on the experimental result, the fatigue behavior of the FRP repaired steel beams with artificial damages was analyzed. The effect of bonding FRP laminates on extending fatigue life of steel beams, the failure modes and the mechanism of fatigue life extension were discussed.

7.4.2 Strengthening Effect

The number of cycles to failure of each specimen is listed in Table 7.3. It was found that the effect of UC laminate in extending fatigue life was much better than that of

SCH11 laminate. The effect of fatigue life extension of beams with side cut strengthened with different laminates is illustrated in Figure 7.17. It is clear that with one layer of UC laminate bonded to the bottom, the fatigue life of steel beam with side damage can be increased more than three times. The effect of fatigue life extension of beams with bottom cut repaired with different laminates is illustrated in Figure 7.18. For the beam repaired with one layer of full length UC laminate, the fatigue life can be enhanced almost twice.

To investigate the effect of different layers of FPR laminates on fatigue life extension of damaged steel beams, three specimens were bonded with one, two and four layers of SCH11 laminates respectively. Figure 7.19 shows the relationship between number of laminate layers and normalized increase of fatigue life. It can be seen that fatigue life of the beams went up with the increase of number of laminate layers in a more or less linear manner. Consequently, the conclusion can be drawn that the increase of number of laminate layers could lead to an extension on the fatigue life. Relationship between number of relative bond length and normalized increase of fatigue life is illustrated in Figure 7.20. The comparison between specimens with 40% and 80% bond lengths shows that the decrease of bond length could shorten the fatigue life of strengthened member. However, the comparison between specimens with eighty percent and full bond lengths indicated there was almost no difference in the effect of bond length on fatigue life extension when the bond length reached certain extent.

7.4.3 Failure Modes

The failure modes of the damaged steel beams repaired with FRP laminates are shown in Figure 7.21. For specimens without strengthening, crack initiated from the tip of the man-made cut. The crack slowly went through the flange and the bottom

of the web. The crack at the other side also appeared and moved towards the web. Finally the deformation of the steel beam increased very fast with the crack tip moving up until collapse. For specimens bonded with FRP laminates, bond failure was not observed before steel cracks appeared. With the crack propagating towards the root of the web, the local bond failure happened around the crack area. After the crack tip reached the root of the web, a global bond failure happened finally and a large part of laminate detached from the steel beam. For specimens with UC laminates, splitting of the laminate could concur with the bond failure as shown in Figure 7.21(d).

7.4.4 Mechanism

To investigate the mechanism of fatigue life extension, finite element analysis was conducted. The FE model for specimen with side damage is shown in Figure 7.22. The material model and the method used in the FE analysis were same as those used in the static analysis. The mesh around the damage area was refined to get detailed stress/strain distribution. Based on the experimental observation, it was noted that the bond failure between FRP laminate and steel beam would not happen until the crack on the beam extended to 20 to 30 mm. The residual fatigue life of the beam at this moment was very limited and negligible. Therefore a perfect bond between the FRP laminate and the steel beam was assumed in the modeling. The comparison of strain distributions of specimen with and without UC laminate is shown in Figure 7.23. It can be seen that the strain distributed around the cut was effectively restrained by the adhesively bonded FRP laminate. The peak strain of the distributed at the tip of the cut was reduced from 0.101 to 0.035 after strengthened by the UC laminate. Therefore, the reduction in the peak strain by the FRP laminate is 65%. The decrease of stress/strain magnitude and stress concentration level was a result of two main effects of the FRP laminates. First, the external bonding of FRP

laminates can increase the stiffness of the beam hence lower stress/strain magnitude in the tension flange. Second, the externally bonded FRP laminate can effectively diminish the local deformation of the area around crack tip. The existence of the external FRP laminate can restrain both in-plane and out-plane relative movement of the two sides around the opening. Therefore, the energy release rates of both mode I and mode II can be effectively reduced.

7.4.5 Concluding Remarks

The feasibility and effect of FRP laminates on extending fatigue life of damaged steel beams was experimentally investigated. Based on the above discussion, the following conclusions can be drawn:

- 1) The fatigue strengthening effect of UC laminate is much better than that of SCH11 laminate. The fatigue life of damaged beams can be increased more than three times by using UC laminate.
- 2) The effect of fatigue life extension can be improved with the increase of number of strip layers and bond length, but effect increased by the later is limited when relative bond length is more than 0.8.
- 3) Premature bond failure will not happen until the crack in the steel member has developed to visible size around 20 mm.

Table 7.1 Specimen details and test results of FRP-steel joints under cyclic loading

Specimen	FRP	Epoxy	P (kN)	V (N/mm ²)	P_N (%)	N_f ($\times 10^3$)
A1	UC	TC	53.9	7.2	100	-
A2	UC	TC	60.4	8.1	100	-
A3	UC	TC	54.5	7.3	100	-
A4	UC	TC	55.4	7.4	100	-
A5	UC	TC	52.0	6.9	100	-
A6	UC	TC	42.5	5.7	70	0.2
A7	UC	TC	42.5	5.7	70	1.8
A8	UC	TC	36.5	4.9	60	31.1
A9	UC	TC	36.5	4.9	60	16.6
A10	UC	TC	30.4	4.1	50	92.3
A11	UC	TC	30.4	4.1	50	28.2
A12	UC	TC	24.3	3.2	40	1070.8
A13	UC	TC	24.3	3.2	40	3924.9
A14	UC	TC	18.2	2.4	30	9981.5
A15	UC	TC	18.2	2.4	30	12094.6
B1	SCH41	S	27.4	3.7	100	-
B2	SCH41	S	24.8	3.3	100	-
B3	SCH41	S	27.9	3.7	100	-
B4	SCH41	S	25.4	3.4	100	-
B5	SCH41	S	25.5	3.4	100	-
B6	SCH41	S	20.2	2.7	70	1.1
B7	SCH41	S	20.2	2.7	70	0.4
B8	SCH41	S	17.3	2.3	60	14.5
B9	SCH41	S	17.3	2.3	60	3.0
B10	SCH41	S	14.4	1.9	50	90.5
B11	SCH41	S	14.4	1.9	50	195.2
B12	SCH41	S	11.5	1.5	40	333.0
B13	SCH41	S	11.5	1.5	40	2429.4
B14	SCH41	S	8.6	1.2	30	11100.3
B15	SCH41	S	8.6	1.2	30	10082.5

Note: P - Ultimate load; V - Shear strength; P_N - Normalized load; N_f - Number of cycles to failure (not applicable to the reference specimens A1~A5 and B1~B5).

Table 7.2 Specimen details and test results of FRP strengthened steel beams under cyclic loading

Specimen	FRP	Epoxy	M (kNm)	P_N (%)	N_f
SA1	UC	TC	46.1	100	-
SA2	UC	TC	36.9	80	275
SA3	UC	TC	32.3	70	1480
SA4	UC	TC	27.7	60	19500
SA5	UC	TC	23.1	50	68000
SA6	UC	TC	18.4	40	1633000
SA7	UC	TC	13.8	30	3165000
SB1	SCH41	S	43.1	100	-
SB2	SCH41	S	34.5	80	25
SB3	SCH41	S	30.2	70	1450
SB4	SCH41	S	25.9	60	12500
SB5	SCH41	S	21.6	50	35000
SB6	SCH41	S	17.2	40	485000
SB7	SCH41	S	12.9	30	3050000

Note: M - Bending moment; P_N - Normalized load; N_f - Number of cycles to failure (not applicable to the reference specimens SA1 and SB1).

Table 7.3 Specimen details and test results of FRP repaired steel beams under cyclic loading

Specimen	FRP	Number of layers	Bond length ratio	Damage	Failure cycles	Increase of fatigue life (%)
R1	-	-	-	Side	9040	-
R2	SCH11	1	1	Side	11085	22.6
R3	SCH11	2	1	Side	14850	64.3
R4	SCH11	4	1	Side	26440	192.5
R5	UC	1	1	Side	36395	302.6
R6	UC	1	0.8	Side	39682	339.0
R7	UC	1	0.4	Side	25874	186.2
R8	-	-	-	Bottom	13122	-
R9	SCH11	1	1	Bottom	14678	11.9
R10	UC	1	1	Bottom	38317	192.0

Note: “Side” refers to the artificial cut on the front and back sides of the bottom flange; “Bottom” refers to the artificial notch on the outer surface of the bottom flange; strengthening parameters (FRP, number of layers and bond length ration) and increase of fatigue life are not applicable to the reference specimens R1 and R8.

Figure 7.1 Test setup and specimen configuration of FRP-steel joint

Figure 7.2 Strain variation along bondline with different load levels

Figure 7.3 Failure mode (a) Z-type rupture (b) U-type rupture

Figure 7.4 Development of strain on laminate with respect to number of cycles

Figure 7.5 Relationship between stress level and number of cycles to failure of specimens in group A

Figure 7.6 Relationship between stress level and number of cycles to failure of specimens in group B

Figure 7.7 Variation of relationship between load and deflection of specimen A12

Figure 7.8 Load-deflection curves for specimens with and without fatigue preloading

Figure 7.9 Setup of FRR strengthened steel beam under cyclic loading

Figure 7.10 Strain development in FRP with respect to number of cycles for SA5

Figure 7.11 Deflections of specimens tested under different load amplitudes

Figure 7.12 Progression of crack length in specimen SA4

Figure 7.13 Debonding of FRP laminate (a) Initiation from the end of FRP laminate (b) propagation along the bondline between FRP laminate and steel beam

Figure 7.14 Relationship between load amplitude and number of cycles in group SA

Figure 7.15 Relationship between load amplitude and number of cycles in group SB

Figure 7.16 Schematic of artificial damage on steel beams (Unit: mm)

Figure 7.17 Extension of fatigue life of beams with side cut strengthened with different laminates

Figure 7.18 Extension of fatigue life of beams with bottom cut strengthened with different laminates

Figure 7.19 Relationship between number of laminate layers and normalized increase of fatigue life

Figure 7.20 Relationship between relative bond length and normalized increase of fatigue life

Figure 7.21 Failure modes of FRP repaired steel beams under cyclic loading
(a) crack in tension flange (b) fracture of steel beam (c) debonding of laminate
(d) splitting of laminate

Figure 7.22 Mesh of finite element model of specimen R1 with side cut at mid-span

(a) Before strengthening

(b) After strengthening

Figure 7.23 Comparison of strain distributions around damaged area before and after strengthening

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

Effective, facile and economical solutions for the rehabilitation and strengthening of steel structures are greatly in need. This research program has made use of FRP composite materials with high strength to weight property for the rehabilitation and strengthening of steel beams. Experimental, numerical and analytical studies on FRP-steel joints and FRP strengthened beams subjected to static and cyclic loading have been conducted. Flexural capacity, cyclic behavior and fatigue resistance of FRP strengthened steel beams, bond performance of FRP-steel joint and debonding strength of FRP bonded steel beams have been investigated.

8.1.1 FRP-Steel Joints

FRP-steel joints with different configurations, material properties, surface treatment methods and bond lengths were tested under monotonic loading. Three different methods including brushing, grinding and sandblasting were used to treat steel surface before bonding of FRP laminates. Test results showed that the joints treated with brushing had lowest bond strength. The bond strength of joints with surface treated by grinding was slightly lower than that of joints with sandblasting treated steel surface, which had the highest bond strength. Joints with stiffer laminates were slightly stronger. Joints with different FRP laminates could have different bond strengths, but the influence of FRP type on bond strength was much less than that of the type of adhesive used for bonding for the materials used in this study. The bond strength of FRP-steel joint is not directly related to the ultimate strength of the

adhesive material. For joints subjected to in-plane loading, the increase of bond strength was not synchronous with that of bond length. There exists a critical bond length beyond which the bond strength will not increase anymore whatever the bond length is.

The failure mode of all types of joints was adhesive failure since the connection between adhesive and steel was the weakest link in FRP-steel joints. The geometry discontinuity at the end of bondline resulted in large deformation in the adhesive to realize force transfer from steel to FRP laminate. Bond failure was caused by the highly concentrated stress in the end of the bondline. For double-lap and tubular joints, the maximum shear stress was much higher than normal stress. For T-peel joint, maximum normal stress was higher than shear stress. For single-lap joint, both shear and normal stresses were quite high.

8.1.2 Flexural Behavior of FRP Strengthened Steel Beams

Flexural behavior of FRP strengthened and repaired steel beams under static loading have been experimentally investigated. For undamaged steel beams, both flexural stiffness and ultimate strength were increased by external bonding of FRP on the tension flange. FRP laminate was found more effective in enhancing the ultimate flexural strength. The high strength of FRP laminate can be better utilized after the steel beam yielding. For most specimens bonded with full length laminates, premature bond failure did not happen before the steel beam underwent large plastic deformation. FRP laminate was found more effective in increasing the flexural stiffness of damaged steel beams. Debonding of FRP laminate happened before the steel beam underwent large plastic deformation. Mechanical clamp at the end of the bondline did not prevent this debonding, which indicated the premature bond failure was caused by the stress concentration at the damaged section. The high strength

property of FRP laminate was able to be better utilized after the yielding of the steel tension flange.

A simple analytical solution was developed for the calculation of flexural capacity of FRP strengthened steel beams. The analysis was based on strain compatibility and equivalent stiffness transformation method thus its application has been limited to FRP strengthened undamaged beams. The analytical solution for flexural capacity of FRP strengthened steel beams was found to be in good agreement with the experimental result although the numerical modeling was able to predict the flexural behavior even better since more accurate material and geometry details can be incorporated into the FE modeling. The flexural capacity of FRP strengthened steel beams increased with the increase of FRP thickness, modulus and bond length but not proportionally. The increase of ultimate strength was much higher than the flexural stiffness when the tensile stiffness of the FRP laminate was small. With the number of FRP layers increasing, the difference between the increases of stiffness and strength became less noticeable. An optimal value of FRP stiffness was found out which can make the steel beam get same increases of flexural stiffness and ultimate strength. The effect of bond length on enhancement of flexural capacity was also less noticeable when a certain extent of bond length was reached.

8.1.3 Bond Performance of FRP Strengthened Steel Beams

Debonding of laminates for strengthened steel beams was studied by performing experiments on specimens with different strengthening parameters. For steel beams bonded with partial-length FRP laminates, premature bond failure might happen before the high strength of FRP can be fully utilized. By experimental exploration, parameters to increase the bond strength were found to include FRP thickness, FRP modulus, adhesive thickness, bond length, spew fillet of epoxy and taper at the end

of the laminate. The premature debonding of FRP laminate can be effectively restrained if the bond length of the FRP is long enough so that the end of laminate is very close to the support of the beam. For strengthened beam with certain length of FRP laminates, the most effective measure to increase the bond strength is to employ the combination of both spew fillet and reversed taper at the end of the bondline.

Based on experimental results of FRP-steel joints and steel beams strengthened with partial-length laminates, a bond failure model to predict bond failure load was proposed. The development of this model utilized the concept of weakest link of material and strain energy density theory. The application of this bond failure model is no longer limited to the hypothesis of elastic material or linear deformation. Geometry details at the end of bondline can also be taken into account in the model. By comparison with the experimental results, the proposed bond failure model was found to be able to predict the bond failure load quite well. By employing a user-subroutine in the FE program, the bond failure model can be incorporated into the numerical modeling to simulate the behavior of FRP strengthened steel beams with bond failure process. With the results from the subsequent parametric study, the relationships between bond strength and different strengthening parameters were found out.

8.1.4 Cyclic Behavior of FRP Strengthened Steel Beams

Bond performance of FRP-steel joints and FRP strengthened steel beams under cyclic loading as well as fatigue strengthening effect of FRP laminates on damaged steel beams was explored in three series of experiments. The bond failure mode of FRP-steel joints under cyclic loading was same as that of statically tested joints. For joints bonded with different types of epoxies, experimental results showed a similar

relationship between load level and number of cycles to failure. For joints tested under the load level of 30% of ultimate strength, bond failure was not observed even after 10 millions cycles of loading and there was almost no stiffness decrease. The bond failure of FRP strengthened steel beams under cyclic loading always initiated from one end of the bondline. The crack between epoxy and steel propagated towards the center of the bondline in a gradual manner. The relationship between load level and number of cycles to failure was similar to that of FRP-steel joints. Thirty percent of static bond failure load can be deemed as the threshold load level for bond failure of FRP strengthened steel beams under cyclic loading. The fatigue life of damaged steel beams can be increased more than three times by using the FRP laminates. The effect of fatigue life extension can be improved with increase of number of laminate layers and bond length. Premature bond failure did not happen until the crack in the steel member developed to a certain extent. The result of FE analysis showed the level of stress concentration can be effectively reduced by external bonding of FRP laminates.

8.1.5 Limitations

As there are many factors affecting the strength, stiffness and bond failure load of FRP-steel joints and FRP strengthened steel beams, the conclusions drawn in this study are subject to the following limitations:

- 1) The conclusion that joints with stiffer laminates are stronger is only applicable for laminates with elastic modulus less than that of steel, since this is true for all the laminates used in this study. For laminates with modulus higher than that of steel, this conclusion is no longer valid.
- 2) Since there are only two types of adhesive materials used in this study, the result of the comparison of influence of FRP and adhesive types on bond strength is only limited to the materials used in this study. For joints with

other different adherend and adhesive materials, the influence of FRP type on bond strength is not necessarily less than that of adhesive type.

- 3) The adhesive failure of FRP-steel joint caused by the concentrated stress in the end of the bondline is the only failure mode observed in this study. For joints with other adherend and adhesive materials, the failure mode and reason to cause the bond failure could be different.
- 4) The address that FRP laminate was found more effective in enhancing the ultimate flexural strength is only valid when the thickness or the elastic modulus of the laminate is low enough. From the parametric analysis it can be seen that when the magnitude of the thickness or the modulus is larger than the optimal value, the increase of the stiffness of will be higher than that of the ultimate strength.
- 5) For FRP strengthened steel beams, the conclusion that the most effective measure to increase the bond strength is to employ the combination of both spew fillet and reversed taper at the end of the bondline is based on the comparison of results with limited parameters involved in this study. If other strengthening parameters beyond the discussion in this study are involved such as the mechanical fastening systems and chemical surface treatment, there would probably be some other measures more effective in enhancing the bond strength.
- 6) Bond strengths of FRP bonded steel structures with different materials could be quite different, since the bond strength is highly dependant on material properties. The bond failure model proposed in this study is only applicable to the FRP strengthened steel structures with same materials used. For such structures with other materials, the deduction procedure used in this study should be repeated by using the same methodology to get a similar bond failure model, which is suitable for the different materials used.

8.2 RECOMMENDATIONS FOR FUTURE RESEARCH WORK

While the contents presented in this thesis have covered several subjects on FRP bonded steel structures including FRP-steel joints, flexural capacity of FRP bonded steel beams, debonding of FRP laminate, cyclic performance and fatigue strength, a great deal of further research work is required to allow further confidence in the system to be developed. Some of these important research topics on FRP strengthened steel structures are discussed below.

1. Strengthening steel structures using prestressed FRP laminates is believed to be a very worthwhile research area. For most CFRP composites, the tensile strength is much higher than that of common low carbon steel. With the rapid development of material science and technology, the ultimate strength of FRP materials is becoming even higher. As presented in this study, the enhancement of stiffness and strength of steel beams before steel yielding was not as remarkable as in the plastic deformation stage. Since FRP materials can have a much longer extension before failure, pretension of laminates, similar to the pretension of steel plate in steel-concrete beams (Albrecht et al., 1995; Uy and Craine, 2004) should be a good measure to bring the high strength of FRP into play. The high strength property of FRP materials can not be intensively utilized otherwise.

2. The effectiveness of strengthening steel beams by bonding FRP laminate to tension flange has been demonstrated. The strengthening effect of FRP on compression component of steel structures is not known yet. An important reason for seldom use of FRP laminate in compression is that under compression FRP laminate is not rigid enough to resist out-of-plane deformation or buckling. However, this disadvantage might be overcome when bonded to steel structures since the rigidity of steel is much higher. If the bond between FRP laminate and

steel substrate is strong enough, strength and stiffness of steel members under compression can also be increased by external bonding of FRP laminates.

3. Durability of FRP strengthening should be investigated to warrantee the use of this technique in severe environment. Research on long-term durability of the bond between FRP laminate and steel structure especially for strengthening with prestressed laminate is necessary before this innovative strengthening method being widespread. Effect of thermal loading is expected to be particularly significant due to the thermal mismatch between the materials. For structures under very high or very low temperature, the thermal effects may be even more significant.

4. There is no standard method to evaluate the bond strength of FRP bonded steel structures with different configuration and geometry. More experimental investigations FRP-steel joints with different material properties, geometries, configurations and loading conditions are needed to systematically study the relationship between bond strength and various parameters and eventually to develop a uniform method to evaluate and predict bond strength. More means to reduce stress concentration level at end of bondline and other measures including mechanical anchor systems to prevent premature bond failure are in need to secure the use of this technique in vital structures.

5. More experiments of FRP strengthened/repared steel structures under cyclic loading are worthwhile to be carried out. Strengthening effect under cyclic loading with different amplitudes, frequencies and stress levels are needed to be examined. Fatigue resistance capacity of bond between FRP laminate and steel with more varying parameters is required to be experimentally explored and more clearly and systemically understood.

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