Retrofit of Steel Structures Using Fiber Reinforced Polymers (FRP): State-of-the-Art

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ABSTRACT
This paper describes the research progress to date in the field of strengthening and repair of steel structures using fiber reinforced polymers (FRP). While this subject has been extensively covered for concrete structures, retrofit of steel structures using FRP has not yet gained the same wide acceptance. This paper provides review of research work on retrofit of steel members including repair of naturally corroded beams, repair of artificially notched beams, strengthening of intact beams, and strengthening of steel/concrete composite flexural members as well as the retrofit efforts of thin-walled tubular sections. The paper also discusses important topics related to the subject such as fatigue behavior, bond and force transfer mechanisms between steel and FRP and the durability of retrofitted systems, particularly the issue of galvanic corrosion. Available field applications are also presented. Research findings have shown that FRP sheets and strips are not only effective in restoring the lost capacity of a damaged steel section but are also quite effective in strengthening of steel sections to resist higher loads, extend their fatigue life and reduce crack propagation, if adequate bond is provided and galvanic corrosion is prevented.

Keywords: FRP, steel, girder, bridge, bond, retrofit, rehabilitation, sheets, strips, carbon, galvanic corrosion.

INTRODUCTION

Steel bridges constitute a large number of the existing bridges worldwide. Corrosion, lack of proper maintenance, and fatigue sensitive details are major problems in steel bridges. It has been reported that 88,000 bridges in the US and Canada are structurally deficient, while the number of functionally obsolete bridges is over 82,000 (*1*). Steel bridges comprise 56 percent of the structurally deficient bridges and almost 44 percent of the functionally obsolete ones. Many steel bridges are in need of upgrading to carry larger loads and increasing traffic volumes. The cost for retrofitting in most cases is far less than the cost of replacement. In addition, retrofitting usually takes less construction time, and therefore, reduces service interruption time. The US DOT has expressed concern about the high number of deficient steel bridges and has enacted the Transportation Equity Act for the 21st century as a way to increase the funding for deficient bridge projects.

Current methods of retrofitting steel bridges and structures typically utilize steel plates that are bolted or welded to the structure. However, constructability and durability drawbacks are associated with this method. Steel plates require heavy lifting equipment and can add considerably more dead load to the structure, which reduces their strengthening effectiveness. The added steel plates are also susceptible to corrosion, which leads to an increase in future maintenance costs. In many cases, welding is not a desired solution due to fatigue problems associated with weld defects (*2*). On the other hand, mechanical details such as bolted connections, which have better fatigue life, are time consuming and costly.

The need for adopting durable materials and cost-effective retrofit techniques is evident. One of the possible solutions is to use high performance, nonmetallic materials such as fiber reinforced polymers (FRP). The superior mechanical and physical properties of FRP materials make them quite promising for repair and strengthening of steel structures.

The use of FRP systems for retrofit of concrete structures has been successful. Its effectiveness has been demonstrated for a variety of retrofitting mechanisms. Today, the use of glass and carbon FRP materials for retrofit of concrete bridges is becoming more widely accepted in practice. FRP is used in the form of sheets or plates attached to the concrete surface for flexural and shear retrofitting or as sheets for wrapping columns to increase their ductility and axial strength. The US Department of Transportation (USDOT) and Federal Highway Administration (FHWA) have sponsored several projects that have led to design guidelines for bridge repair and both have implemented these retrofitting schemes into several projects.

Bonding of FRP materials to metallic structures was first used in mechanical engineering applications. Carbon fiber reinforced polymer (CFRP) laminates have been successfully used to repair damaged aluminum and steel aircraft structures (*3*, *4*). Bonding of composite laminates was also shown to have many advantages for marine structures (*5*, *6*). For civil engineering structures, previous work conducted on the strengthening of metallic structures using CFRP has been focused in three main areas: strengthening of iron or unweldable steel girders, rehabilitation of corroded steel girders, and repair of fatigue damaged riveted connections.

The low-tensile modulus of glass fiber composites (GFRP), ranging between 72 GPa, and 87 GPa, makes them less desirable for retrofitting steel structures. On the other hand, CFRP displays outstanding mechanical properties, with a typical tensile strength and modulus of elasticity of more than 1,200 MPa and 140 GPa, respectively. In addition, CFRP laminates weigh less than one fifth of the weight of a similar size steel plate and are also corrosion resistant. CFRP plates or sheets can be bonded to the tension face of the member to enhance its strength and stiffness as shown in Figure 1(a). By adding CFRP layers, the stress level in the original member will decrease, resulting in a longer fatigue life.

When considering the retrofit of steel structures using FRP materials versus retrofit using steel plates, there are two considerations that favor FRP materials. First, the costs associated with retrofitting are often more associated with time limitations for completing the project, as well as labor costs and the costs to divert traffic, and to a lesser extent, with material costs. Due to the light weight of FRP composite materials, it is expected that they could be installed in less time than by strengthening with the equivalent number of steel plates. The second factor that favors composites, especially CFRP, is its higher tensile strength in comparison to the yield strength of steel, provided that adequate means of bonding are introduced.

A case study was performed to examine the economic advantages of rehabilitation of damaged steel girders with CFRP pultruded laminates as compared to replacement of the girders (*7*). In Delaware, bridge girders, with a total length of 180 meters, were replaced due to severe and extensive damage. The replacement costs were compared with the cost of rehabilitation at an assumed 25 percent section loss. It was concluded that total replacement cost was 3.65 times higher than the cost of rehabilitation (*7*).

The following sections address different aspects of retrofit of steel structures using FRP materials in light of the available and published literature review.

RETROFIT OF STEEL GIRDERS

Research efforts to examine the feasibility and efficiency of retrofit of steel girders have been mainly conducted using one or combination of the following approaches:

- Repair of naturally deteriorated steel girders.
- Repair of an artificially notched girder to simulate fatigue cracks or section loss due to corrosion.
- Strengthening of an intact section to increase the flexural strength and stiffness.
- Retrofit of steel girders in composite action with a concrete deck.

Repair of Naturally Deteriorated Steel Girders

Four full-scale girders were removed from an old deteriorated bridge (*7*). The four corroded bridge girders were 9754 mm in length, 610 mm deep, and had a flange width of 229 mm. All girders had uniform corrosion along their length. The corrosion was mostly concentrated on the tension flange, as is typical for many bridges. The evaluation of the girders indicated approximately 40 percent loss of the tension flange. This flange loss resulted in a 29 percent reduction in the stiffness. Since the webs of the girders were not severely corroded, it was decided that only the bottom flanges would be reinforced, along the entire length of the girders, with a single layer of CFRP strips. The CFRP strips used were 6.4 mm thick and 38.1 mm in width. Two of the rehabilitated girders were tested under cyclic loading within the elastic range up to a load approximately 50 percent of the yielding load of the unrepaired girder. After two cycles, the girders were loaded until local buckling failure occurred in the compression flange. It was found that the CFRP retrofit increased the elastic stiffness of the girders in the range of 10 to 37 percent. The corrosion of the first girder was more severe than that of the second girder. Results showed that the ultimate capacities of the first and second girders were increased to 17 and 25 percent respectively, over the predicted capacities of the unstrengthened specimens. Furthermore, the inelastic strains in the tension flange were reduced by 75 percent in comparison to the unreinforced girder at the same load level.

Repair of Notched Girders

Experimental work was done at the University of Missouri-Rolla (*8*), where three-point bending tests were conducted using four W12 x 14 simply supported girders with a span of 2438 mm. The first specimen was tested without retrofit, to serve as the control specimen. The second specimen was also tested without FRP retrofit, however a 106 mm wide notch was cut into the tension flange to simulate a severe loss of section due to corrosion. The third and the fourth specimens were also notched similar to the second specimen and were retrofitted with 100 mm wide CFRP laminates that covered the entire length of the third beam and covered one quarter of the beam length of the fourth beam, to examine the effect of the bond length. Four pairs of lateral supports were used to reduce the effect of lateral torsional buckling and were located at both the support locations and the quarter-span locations.

Specimens 1, and 2 were failed by lateral buckling. However, the failure mode of specimen 3 (full retrofit) was due to debonding of the CFRP laminate, which initiated at the notch, located at the mid-span of the girder, and then extended to the end of the CFRP laminate with increasing the applied load. This behavior was triggered by the high stress concentration and high shear stresses near the notch area. Failure of specimen 4 was due to sudden debonding of the CFRP laminate. Test results indicated 60 and 45 percent increase in the plastic load capacities of specimens 3 and 4, respectively.

Tavakkolizadeh, and Saadatmanesh (*9*) 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 research considered different CFRP lengths for each group of notches (100, 200, and 300 mm for the shallow notch and 200, 400, and 600 mm for the deep notch). The results indicated that stiffness and ultimate load carrying capacity of the repaired beams were close to its 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. The main difference of the results for deep cuts compared to shallow cuts was the significant loss of ductility. It was reported that this could be overcome by using relatively long patches.

Although the method of notching the tension flange of flexural members is efficient in reducing the capacity of the specimen, the authors believe that such a technique does not reflect damage and deterioration conditions in the field quite accurately. It is rather more representative to use beams, which are naturally corroded or beams subjected to fatigue. In this case, the section loss due to either corrosion or fatigue cracks is randomly distributed over a longer segment of the span and over the bonded length of FRP, which could impact the bond strength of the FRP as compared to that measured using a notched beam.

Shulley et al. (*10*) studied the validity of using FRP sheets to repair the damaged web of steel beams. Six beams of 711 mm span were tested using three-point bending, where a 100 mm diameter hole was drilled at the mid-height of the web, 178 mm away from the support, to simulate web damage. Different types of FRP sheets were used in the repair procedure. All patched beams failed in a similar fashion. As the load was increased, the patch began to buckle over the area where the hole was located, then separated from the web. Therefore, all the repaired beams could not achieve even the strength of the undamaged beam. It should be noted that the fiber orientations in the different FRP patches were not reported. The authors believe that the fiber orientation with respect to the directions of principal stresses in the web would have a significant impact on the strength of the patches.

Strengthening of Undamaged Girders

Four different repair schemes were applied (*11, 12*) to the tension flange of W8x10 steel beams of 1372 mm span, to demonstrate the advantages of using FRP in strengthening of steel girders. A schematic of the basic reinforcement geometries used is shown in Figure (1).

The composite reinforcement was applied over the center 1219 mm of each beam. The first scheme consisted of a 4.6 mm thick CFRP plate, bonded directly to the tension flange as shown in Figure 1(a). The second reinforcement scheme utilized a similar CFRP plate, but was attached to an aluminum honeycomb structure to position the CFRP plate further away from the steel section (sandwich construction) to increase the moment of inertia of the section as shown in Figure 1(b). In the third scheme, a different strategy was developed. A foam core was attached to the tension flange, followed by wrapping the whole assembly by a GFRP sheet as shown in Figure 1(c). The fourth scheme utilized a GFRP pultruded channel, which was both adhesively bonded and mechanically connected to the tension flange with self-tapping screws as shown in Figure 1(d). Four-point bending tests were performed on the specimens. The increase in stiffness was 20, 30, 11 and 23 percent for the schemes shown in Figure 1(a, b, c, and d) respectively, while the increase in strength was 42, 71, 41 and 37 percent for the same schemes. It was concluded that the sandwich CFRP-plated technique, Figure 1(b), is the most efficient, while the GFRP wrapped, Figure 1(c), is the least effective.

Abushaggur and El Damatty (*13*) studied the effect of bonding 19mm GFRP plates to increase the capacity of a W6x25 steel cross section. Four-point bending tests were conducted on beams of 2800 mm span. GFRP plates of 2400 mm length were bonded to both the top and bottom flanges. The reported modes of failure were either rupture of GFRP plates or delamination between GFRP layers, but with no failure of the adhesive between steel and GFRP. The increase in stiffness, yield and ultimate loads was 15, 23 and 78 percent respectively.

In the previous studies (*12* and *13*), a 4.6 mm thick CFRP plate, and a 19 mm thick GFRP plate were bonded to steel girders to enhance the structural performance. The GFRP plate, which is four times thicker than the CFRP plate, has provided an increase in the stiffness, 25 percent lower than that provided by CFRP. On the other hand, the GFRP plate has provided an increase in the strength, 86 percent higher than that provided by CFRP. It is therefore recommended that future feasibility studies be conducted to compare the effectiveness of GFRP and CFRP systems in enhancing both the stiffness and strength, especially with GFRP has the advantage of lower cost and also does not lead to galvanic corrosion problems.

Retrofit of Steel Girders Acting Compositely with a Concrete Deck

The presence of a concrete slab in composite action with steel girders provides a continuous lateral support that prevents premature torsional buckling. Furthermore, the location of the neutral axis of a composite section, utilizes strengthening applied to the tension flange more effectively. Consequently, the tensile strengths of both the steel flange and the attached CFRP retrofit system control the overall strength of the beam.

Sen et al. (*14*) tested two groups of specimens with yield strengths of 310 MPa and 370 MPa. All specimens were 6100 mm long W8x24 steel beams in composite action with a 114 mm thick and 710 mm wide reinforced concrete deck. Four-point bending tests were first performed on all specimens to simulate severe service distress by introducing yield stresses in the tension flanges. The damaged specimens were then repaired by bonding 165 mm x 3650 mm CFRP laminates of 2 and 5 mm thickness to the tension flange and then tested to failure.

Two tests with the 2 mm thick laminate were stopped after their deflection exceeded the 100 mm limit of the LVDT. The tests of specimens with 5mm laminates were ended suddenly when the CFRP laminate separated from the steel flange. The increase in ultimate strength was found to be between 46 and 109 percent. The smaller increases occurred in the specimens retrofitted with the 2 mm laminates, while the larger increases were observed in the specimens retrofitted with the 5 mm laminates. In addition to increasing the ultimate load, the elastic region of the sections was considerably extended in the repaired specimens, from about 20 to 67 percent.

Tavakkolizadeh, and Saadatmanesh (*15*) performed both experimental and analytical studies on three composite girders using W14x30 steel beams with 355 MPa yield strength. A 75 mm thick and 910 mm wide concrete slab was attached to the upper steel flange. Two sets of 75 mm wide and 1.27 mm thick CFRP sheets were placed side by side and bonded to the girder. The sets included 1, 3, and 5 layers of sheet. Four-point bending tests were then performed on the 4780 mm long specimens. The behavior of one of the retrofitted girder is shown in Figure (2) compared to the behavior of the virgin girder. 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. Furthermore, The tensile strains in the flanges, at a given load, decreased by an average of 21, 39, and 53 percent for the three different retrofitted girders, compared to the virgin girder. It has been shown also that the stress in the laminates at failure drops from approximately 75 percent of its ultimate strength for the one-layer system to 42 percent in the five-layer system. 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.

Based on the previous studies (*14* and *15*), test results are plotted by the authors in Figure (3) to show the effect of the CFRP reinforcement ratio on the flexural strength gain for beams with different yield strengths. The reinforcement ratio is defined as the ratio of the area of CFRP to that of the steel section and accounts for the number of the CFRP layers. The figure shows the increase in flexural strength as the reinforcement ratio is increased. Additionally, the strength increase is higher for steel with lower yield strength. It should also be noted that the strengthening effectiveness is reduced for thicker laminates as the failure is governed by debonding rather than by fracture of FRP.

RETROFIT OF TUBULAR STEEL SECTIONS

Vatovec et al. (*16*) tested rectangular steel tubes retrofitted with different configurations of 50 mm x 1.2 mm CFRP strips, attached to the tension and compression flanges, using simple beam tests. To avoid local buckling of the tubes' upper flange, the middle half of the specimens' length was filled with concrete. Test results showed that the ultimate moment capacity was increased from 6 percent for the tube reinforced with one strip attached to the compression flange, to 26 percent for the specimen that had two strips attached to the tension flange and one strip attached to the compression flange. The governing failure mode of all specimens was delamination of the CFRP strips on the compression flange, followed by the delamination of the strips on the tension flange. The CFRP strips on the compression flange buckled upwards, splitting longitudinally, and then fracturing across a portion of the width. Adhesive was present on both CFRP strips and steel tubes.

Toutanji, and Dempsey (*17*) performed a theoretical study on the effect of winding three different types of FRP sheets, including glass, aramid, and carbon, around damaged steel pipelines. A mathematical model was developed to predict the stress levels induced due to the effect of soil loads, traffic loads, and the internal pressure. The results of the analysis showed that carbon fiber sheets provide better performance than glass or aramid in improving the internal pressure capacity of pipes at ultimate stress.

FATIGUE BEHAVIOR OF STEEL SECTIONS RETROFITTED WITH FRP

The use of steel plates to repair and strengthen existing steel structures has been traditionally used for rehabilitation of steel girders. However, the welded detail of steel plates is sensitive to the fatigue loads. Various researchers have examined the effectiveness of using bonded CFRP sheets or plates to improve the fatigue strength.

Gillespie et al (*18*) conducted fatigue testing on two strengthened girders, which were removed from an old bridge (*7*) as indicated earlier. The two specimens were tested in fatigue under 10 million cycles, within the stress range expected in the field. Throughout the 10 million cycles, the CFRP plates were periodically monitored and inspected for debonding. No evidence of CFRP plate debonding was found.

Bassetti et al (*19*) tested the effect of bonding prestressed CFRP plates on reducing the rate of crack propagation and increasing the fatigue life of riveted steel structures. Two research programs were conducted on both small- and full-scale specimens. CFRP plates of 1.2 mm thickness were attached to central-notched specimens. The specimens were loaded with a stress range of 80 MPa and a stress ratio of 0.4. The results showed that the crack growth rate was drastically decreased and the fatigue life was increased by a factor up to twenty, depending on the prestressing level. The authors, however, did not report the details of the prestressing technique or procedure.

Full-scale tests were also carried out on cross girders 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 MPa and a stress ratio of 0.10. After strengthening the cross girder with five CFRP plates, no crack growth was recorded up to 20 million cycles.

Buyukozturk et al. (*20*) also performed an experimental study involving fatigue testing of side-notched steel specimens repaired with FRP patches of various widths and lengths. The study has confirmed the effectiveness of the technique by increasing their fatigue lives as the width and length both increase. It was found that the fatigue life of notched specimens, which would fail under applied stress in the unrepaired configuration, could be significantly increased with the application of FRP composites.

Tavakkolizadeh, and Saadatmanesh (*21*) studied the fatigue strength of 21 specimens made of S127x4.5 (SI Designation) steel beams. The clear span of all specimens was 1220 mm and were all tested under four-point bending with 200 mm spacing between the loading points. Both edges of the tension flange were cut at mid-span using a band saw with a blade thickness of 0.9 mm. The length of the cuts was 12.7 mm. CFRP sheets, 300 mm in length, and covering the full width of the flange were then bonded to the lower face of the tension flange. For all stress ranges considered in this study, this technique improved the fatigue life of the detail by about 2.6 to 3.4 times that of the unretrofitted specimens. This improvement was equivalent to upgrading the detail from AASHTO category D to category C (*22*). Furthermore, for the same crack length, the retrofitted specimens were stiffer than the unretrofitted specimens.

Mosallam et al (*23*) studied the effect of strengthening steel frame connections using CFRP stiffeners to resist cyclic loads. Two strengthening details were investigated, including an adhesively bonded CFRP stiffener and a mechanically fastened CFRP stiffener. The effectiveness of the two techniques was compared to that of a fully welded control specimen. Test results indicated that the proposed techniques achieved acceptable ductility ranges. The CFRP repair technique provided the highest ductility with an increase of more than 1.25 times the ductility of the fully welded control specimen.

SURFACE PREPARATION AND BONDING OF FRP

Surface preparation is the key to a strong and durable adhesive bond. Since rehabilitation takes place on-site, surface treatment must also be environmentally friendly, and easily accomplished in field conditions. Brockmann (*24*) has shown that application of the CFRP material can occur up to 150 hours after completion of the surface preparation. If strengthening occurs after this time, a lower bond strength could result.

Surface grinding or sandblasting is recommended to remove all rust, paint, and primer from the steel surface. Additionally, the bare steel may be pretreated using either an adhesion promoter or a primer/conditioner, which leaves a thin layer attached to the metal oxide surface (*25*). This type of bond significantly improves the long-term durability because water displacement through this coating is unlikely since the hydrolysis of the primary bonds is a slow process. The bonded side of the FRP plates may be sanded to increase the surface roughness using medium grit sandpaper or a sandblaster, and wiped clean with acetone. However, excessive surface preparation of FRP plates may expose the surface of the carbon fibers leading to possible galvanic corrosion if placed in direct contact with the steel surface. The adhesive is then applied to the pretreated steel surface, bonding either FRP laminates or sheets to the steel. The adhesive typically used is a two-component viscous epoxy. A less viscous epoxy is used for bonding the laminates to each other. It is recommended to leave the bonded plates to cure for a sufficient time, not less than 48 hours. Miller et al. (*18*) suggested application of an accelerated curing method, such as heating blankets or induction heater to increase the curing rate of the adhesive.

An adhesive for a particular rehabilitation scheme must perform three functions. First, the adhesive must have adequate bond strength so that the composite material can be optimally utilized. Consequently, this requires the failure mode of the system to be governed by the ultimate strength capacity of the composite and not by a premature bond failure. Second, the system must be sufficiently durable in the design environment to match the life expectancy of the structure (typically 75 years). Finally, the adhesive must also be easy to utilize under field conditions.

Force Transfer

Force transfer between FRP and steel takes place through bond at the interface between the two materials, which is influenced by several factors including bonded length, types of fiber and resin, surface preparation, thickness of adhesive and thickness of FRP laminate. Experimental and analytical studies were performed by Miller (*26*) to quantify the force transfer of a 457 mm CFRP plate bonded to the tension flange of a steel girder. It was found that approximately 98 percent of the total force transfers within the first 100 mm of the end of the bonded plate. Therefore, it was determined that the development length was on the order of 100 mm for this type of plate and adhesive.

Analysis has also shown that the epoxy failure at the ends of the FRP laminates or plates is due to high peeling stresses normal to the surface. Abushaggur and El Damatty (*13*) developed a finite element model to predict the distribution of peeling stresses for a beam subjected to four-point bending. The distribution shows a symmetric behavior about the center of the beam. It was found that the critical locations for peel off failure are towards the edges of the FRP sheets.

In order to prevent this type of failure, different techniques have been proposed. Vinson (*27*) stated that beveling the CFRP plates to a 45^{o} angle at all terminations could effectively limit the peeling stresses. Sen et al. (*14*) designed a steel clamp and positioned it precisely over the ends of the laminates to withstand the predicted peeling stresses. The proportions of the clamp were made larger than the width of the steel flange so that the assembly could be held in place by bolts without necessity of drilling holes in the steel flange or the FRP laminates. Furthermore, bolts could be used to augment the load transfer capacity of the epoxy adhesive, especially with thicker laminates. Liu et al. (*8*) suggested wrapping GFRP sheets around the tension flange and part of the web perpendicularly to the longitudinal laminates. These sheets would be applied along the length of the girder to avoid delamination of the CFRP laminates.

Kennedy (*28*) studied the effect of bonded CFRP patches on the flow of load through steel plates with an internal through-thickness crack. The study showed that the load flowed through the CFRP patches; on the patched face; across the crack and transfer quickly back into the steel. However, the load flow in the unpatched face is much the same after patching as before. The study showed also that one-sided patching decreases crack tip strains significantly in the patched face and increases them slightly in the unpatched face.

DURABILITY OF STEEL MEMBERS RETROFITTED WITH FRP

One of the most important factors affecting durability is the environmental surroundings. The FRP retrofitting system itself is non-corrosive, however, when carbon fibers become in contact with steel, a galvanic corrosion process may be generated. Three requirements are necessary for galvanic corrosion to occur between carbon and steel: an electrolyte (such as salt water) must bridge the two materials, there must be electrical connection between the materials, and there must be a sustained cathodic reaction on the carbon (*29*). By eliminating any one of these requirements, the galvanic cell is disrupted. A good selection of adhesives with inherent durability and high degree of resistance to chlorides, moisture, and freeze-thaw cycles is also very important.

In order to test the durability of the bond between the composite and steel, tests were conducted using the wedge test (*30*). This test has great sensitivity to environmental attack on the bond between materials. The test was proved to have a very high degree of correlation with service performance and is considered more reliable than conventional lap shear or peel tests (*30*).

Shulley et al. (*10*) performed wedge tests on five different types of reinforcing fibers (Three types of carbon fibers, and two types of glass fibers.). Specimens were placed in five different environments (hot water, freezer, freeze/thaw, salt water, and room temperature water.) for two weeks before the initiation of the wedge test. After the wedge was inserted into the bond line, the specimens were returned to their respective environment. The recorded crack growth rate after seven days showed no dominance of one system over the others. Evidence that the GFRP reinforced systems have a more durable bond with steel than the CFRP based systems was apparent. However, the result that one of the carbon fiber specimens exhibited the smallest crack growth rate upon exposure to seawater indicated that no galvanic corrosion occurred during the time interval studied. Also, the most durable bonds were those subjected to a subzero environment. Karbhari and Shully (*4*) suggested using a hybrid of glass and carbon, with the glass being in contact with the steel, to achieve both durability and performance criteria.

Brown (*31*) studied the corrosion of CFRP bonded to metal in saline environments. The metals investigated included aluminum, steel, stainless steel and titanium. Specimens were fabricated by either bolting the CFRP laminate plate to the metal or by bonding with epoxy. Accelerated testing was performed by placing the specimens in a continuous fog of neutral sodium chloride solution at a temperature of $35^{o}C$ for 42 days. It was found that for all the metals studied, there was no accelerated attack due to galvanic coupling for the specimens that were adhesively bonded. However, considerable attack occurred for the bolted specimens. Since most structural adhesives are insulators, and provided that a continuous film of adhesive can be maintained over the bonded region, there is no possibility for galvanic corrosion.

Numerous studies have also been conducted on aluminum and steel structures reinforced with CFRP for aerospace and marine applications. Coupling CFRP with aluminum should be a more critical test for durability since the electrode potential between carbon and aluminum is even greater than the potential between carbon and mild steel (*29*). Armstrong (*3*) described a technique for patching cracks on aluminum passenger aircraft used in the commercial fleet. A repair that was used for 20 months on the leading edge of the wing on the Concord and had flown for 2,134 hours and was subjected to 576 supersonic flights had to be chiseled off and removed. It was noted that this repair appeared to be well bonded over the entire patched area. For repairs of ships with CFRP patches, Allan et al. (*5*) used a moisture barrier comprised of an additional foil sheet and chopped strand glass laminate to cover the structural CFRP patch. In addition to the electrical isolation of the carbon fiber from the metal structure by the resin matrix, two of the three conditions for galvanic corrosion were controlled.

Galvanic corrosion, has also been investigated by Tavakkolizadeh, and Saadatmanesh (*32*) The results of the experiments showed that when steel and carbon fibers coated with a thin film of epoxy are coupled together, the corrosion rate of steel increases by a factor of 24 and 57 in a deicing salt solution and seawater, respectively. It was shown also that the corrosion rate is directly related to the epoxy coating thickness. Therefore, applying a thin film of epoxy coating (0.1 mm) decreased the corrosion rate in seawater sevenfold. However, by applying a thicker epoxy coating (0.25 mm) the corrosion rate was decreased by twenty-one times. Several techniques are recommended for the elimination of galvanic corrosion, including the use of a nonconductive layer of fabric between the carbon and the steel, an isolating epoxy film on the steel surface or a moisture barrier applied to the bonded area.

In order to prevent the expected galvanic corrosion completely, a corrosion barrier (glass fabric layer described earlier) is placed between the CFRP plate and the steel during the bonding process. Several studies by West (*33*) have shown this procedure to be effective in preventing galvanic corrosion. However, care must be taken such that the placement of glass fibers does not lead to blistering of the composite. Tucker and Brown (*34*) found that glass fibers placed into a carbon fiber weave appeared to promote blistering of the composite by creating conditions favorable for osmotic pressure to be developed. Clearly, water held within the bond line by osmotic pressure is not favorable for maintaining a durable bond.

Another issue involving the fatigue resistance of the bond was studied in the University of Delaware. The retrofit was viewed as having good fatigue resistance even with large number of load cycles. Furthermore, Bassetti, et al (*35*) studied the influence of varying the type of epoxy and the curing temperature on the fatigue behavior. Test results showed no significant difference in fatigue life, or behavior.

FIELD APPLICATIONS

Field installations demonstrate that the rehabilitation technique can be applied under actual field conditions. The rehabilitation and associated pre- and post-field diagnostic testing allow for further evaluation of the effectiveness of the retrofit system in providing stiffness and strength increases for structures.

The 1-704 bridge, which carries southbound I-95 traffic over Christina Creek, just outside of Newark, was chosen by Delaware Department of Transportation to assess the CFRP rehabilitation process conducted by the University of Delaware. One layer of CFRP plates was bonded to the outer face of the tension flange of the steel girder, which has a span of 7500 mm and a W24x84 cross section. Six CFRP plates were placed side-by-side to cover the entire flange width. The CFRP plates were installed over the full length by using four overlapped sections. Each section was 1500 mm long and had staggered joints. At the joints between the plate sections, a 300 mm stagger of every other CFRP plate was used. Consecutive CFRP plates were beveled at a 45o angle to form a scarf joint instead of a typical butt joint. Load tests were performed on the chosen girder, prior to and after the rehabilitation. A comparison between the load test data indicated that adding a single layer of CFRP plates resulted in 11.6 percent increase in girder stiffness, and 10 percent decrease in strain.

Two historic metallic bridges in the UK were also strengthened with CFRP plates (*36*). The first bridge was the Hythe Bridge, which had eight inverted Tee sections (cast iron beams) of 7800 mm span. Four prestressed CFRP plates were bonded to each beam by epoxy adhesive in addition to the end anchorages. The prestressing level was designed to remove all tensile stresses. The second bridge was Slattocks. The bridge beams were 510 mm deep and 191 mm wide, and supported a reinforced concrete deck. CFRP plates of 8 mm thickness were bonded to the bottom flanges of 12 innermost beams. A feasibility study was done and indicated that it would have cost much more to install a set of special traffic lights to control vehicle flow for traditional bridge repairs as it has for the total strengthening work using CFRP plates, where the traffic was allowed to keep moving over the bridges during the strengthening process.

CONCLUSIONS

Research interests in the field of retrofit of steel structures using FRP materials are gradually increasing. Although using FRP for retrofit of steel structures has not yet gained the same popularity and wide spread use as in concrete structures, the literature to date shows positive and promising evidence of success. The following conclusions could be drawn:

1. The use of FRP sheets and strips is not only effective for restoring the lost capacity of a steel section, as a repair technique, but is also quite effective in strengthening of steel structures to resist higher loads.
2. Epoxy bonded FRP sheets and laminates are quite promising in extending the fatigue life of steel structures. The FRP has a significant effect on reducing the crack propagation.
3. Strengthening using FRP results in increasing the yielding load of the steel section. Consequently, the service load can be increased.
4. The galvanic corrosion may be initiated when there is a direct contact between steel and CFRP, the steel and the CFRP are bridged by an electrolyte and there is a sustained cathodic reaction on the CFRP. Precautions can be taken to eliminate this problem by using a nonconductive layer between the carbon and steel or by protecting the area from moisture ingress.
5. Delamination of FRP in the compression side of the girder could occur before delamination in the tension side due to buckling of the laminate. Therefore, bonding the FRP reinforcement to the compression side may not be as effective as bonding it to the tension side.
6. The lower value of modulus of elasticity of all currently available FRPs, including CFRP, in comparison to steel, may result in increasing the number of layers required to increase the stiffness of the section and consequently could affect the cost effectiveness of such technique.
7. As the number of FRP layers increases, the efficiency for utilizing the full strength of the FRP material decreases, since the stress in the FRP laminate for one layer was much higher than that for multiple layer system. It has been shown that the thicker the reinforcing material, the higher the chance of bond failure. Consequently, balanced design should be considered to effectively utilize the strength of CFRP laminates.
8. Strengthening the tension flange of I-sections with FRP will result in increasing their moment capacity. Consequently, lateral torsional buckling of the compression flange may control the failure. Therefore, such strengthening technique is more effective when sufficient lateral supports of compression flange are provided as in composite sections.
9. Applying prestressing force to CFRP plates is very efficient in retrofitting of steel structures subjected to fatigue loads. It prevents further cracking by promoting crack closure effect, which increases the stiffness of the cracked sections.
10. Four-point bending tests show small influence of the FRP bond length on the delamination failure mode, especially when the entire bonded length of FRP is located within the constant moment zone. Three-point bending tests have shown the importance of the bond length on this particular mode of failure, due to the presence of shear stresses along the entire span of the beam.

RESEARCH NEEDS

Due to the promising performance and other advantages of bonding FRP laminates to steel structures, this technique is becoming increasingly popular. However, there are still many issues that need to be investigated. A number of potential research needs can be noted:

1. Research efforts need to be focused on two promising applications of FRP in retrofit of flexural steel members, namely increasing the stiffness under service loads in order to reduce the deflections, and increasing the ultimate moment capacity of the section.
2. In order to increase the strength of the member, various bonding techniques need to be investigated further to avoid the premature FRP delamination, and consequently increase the retrofitted member's capacity and maximize the benefits of the superior FRP properties. These techniques could be based on adhesive or mechanical bonding.
3. For retrofit applications of bridge girders, more research is needed on composite sections, where the concrete slab has a significant effect on the flexural performance of the steel girders.
4. Further research is needed to develop low cost carbon fibers/resin system with superior strength and stiffness characteristics compared to conventional CFRP, in order to decrease the number of required layers. High modulus CFRP materials with superior stiffness could be a good alternative.
5. Further studies on the long-term performance of bonded FRP are needed, particularly the issue of galvanic corrosion and the effects of different thermal coefficients of FRP and steel on the bond strength and durability.
6. The literature lacks research on retrofit of compression steel members, particularly closed tubular and square sections. It is anticipated that FRP could possibly delay the local buckling of thin-walled sections.
7. In general, further research is needed to produce sufficient experimental data in order to develop design guidelines for strengthening steel members using FRP materials.

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LIST OF FIGURES

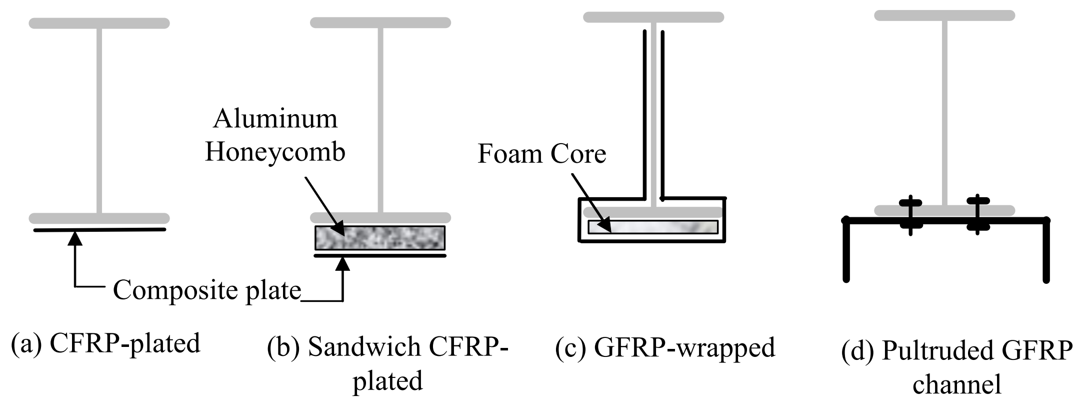

Figure (1) Different Retrofit Schemes

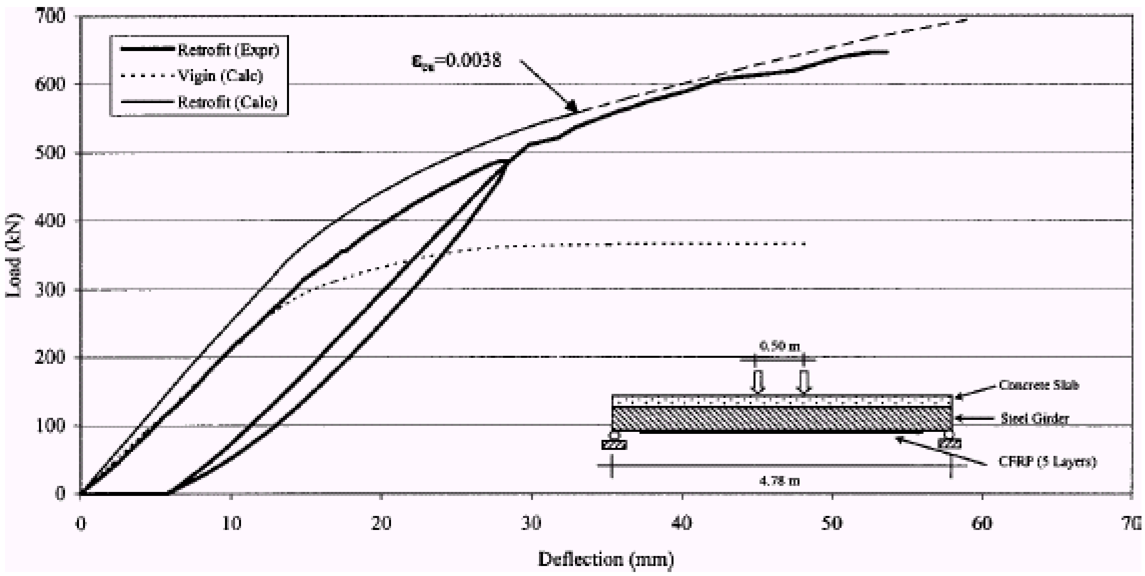

Figure (2) Load-deflection behavior of girder retrofitted with five layers of CFRP sheet.
(Adopted from Reference (*15*))

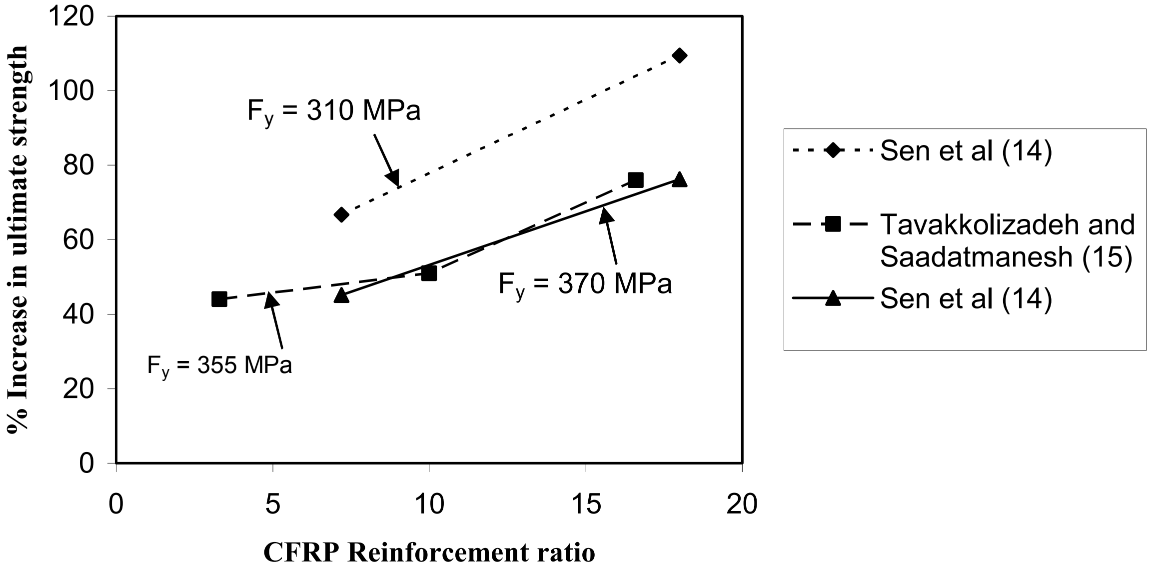

Figure (3) Effect of CFRP reinforcement ratio and yield strength on the ultimate strength of retrofitted steel girders.