Rehabilitation of Steel Bridge Members with FRP Composite Materials

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**ABSTRACT:** Throughout the US there are thousands of steel bridges that are at various levels of advanced deterioration due to many years of service and exposure to the environment. When total replacement is not an option and traditional retrofit methods are not economical and time consuming, an alternative retrofit method using FRP composite materials provide engineers with a very competitive solution that will increase the life-cycle of these steel bridges. A strengthening method using fiber reinforced polymer (FRP) material for repairing corroded steel members is discussed in this paper.

1 INTRODUCTION

The traditional retrofit method for repairing corroded steel members is to attach steel plates to these members through bolts or welding. However, there are many disadvantages associated with this method, such as; (1) the procedure is labor intensive and time consuming, (2) it requires drilling and extensive lap splice detailing, (3) traffic must be closed for an uncertain period of time, (4) there is a potential for weld fatigue cracking at the cover plate ends, (5) increase in the weight of the members, which may lead to deficiency in the member capacity and increases in deflections.

The concept for retrofitting corroded steel members with FRP materials is to strengthen these members with laminates that are adhered to the steel surface with an epoxy paste. This method has many advantages, such as; (1) it is rather easy and simple to implement in the field, (2) traffic disruptions are maintained at minimum levels, (3) it is durable (Nouredine, 1996), (4) it may allow the full recovery of the stiffness and load capacity of the corroded members without exceeding significantly their initial weight (Nouredine, 1996). These advantages typically offset the initial cost associated with FRP materials.

2 TEST SETUP

This paper presents the work in progress associated with this research program. The main objective of this research program is to develop competitive techniques capable of retrofitting corroded steel members to within its original ultimate flexural strength. In this series of tests, four beams were tested to develop a simple/economic concept for FRP retrofit of steel girders.

### Table 1. Property of the S&P Laminates

<table>
<thead>
<tr>
<th>Property</th>
<th>Metric System</th>
<th>SI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity</td>
<td>&gt;29 Msi</td>
<td>&gt;200 GPA</td>
</tr>
<tr>
<td>Tensile strength at break</td>
<td>&gt;360 ksi</td>
<td>&gt;2300 N/mm²</td>
</tr>
<tr>
<td>Width/thickness</td>
<td>3.94in./0.055in.</td>
<td>100mm/1.4mm</td>
</tr>
<tr>
<td>Maximum Elongation</td>
<td>37.1/49.5 k-lb</td>
<td>168/224×10⁻³ N</td>
</tr>
</tbody>
</table>

Four W12×14 girders were selected for the first series of tests. S&P laminates CFK 200/2000 obtained from “Structural Composites Inc.” were chosen as the strengthening materials. The property of the FRP material is provided in Table 1.

The girders in this test series were tested in the Engineering Research Laboratory at University of Missouri-Rolla (UMR). Loading of the girders was applied at their mid-span (i.e. three-point bending test). A typical compression machine was used for the load application. The total length of these girders was 108 in. (274.32 cm), with the length between the two vertical supports of 96 in. (243.84 cm). This test setup is shown in Figure 1 and the test matrix for this series of tests is shown in Table 2.

All the four test units were tested in a three point bending test. Test unit 1 was designed with no retrofit or notch to serve as the control test unit. Test unit 2 was designed with no FRP retrofit, but a 4 inch wide notch was imposed in the tension/bottom flange to simulate the severe loss of section due to corrosion. Tests unit 3 and 4 were upgraded with 3.94 in. (100 mm) wide CFRP laminate that covered the length of the beam and one
quarter of the beam length, respectively, to study the required bond length.

Figure 1. Test Setup

Table 2 Test Matrix

<table>
<thead>
<tr>
<th>Test Unit</th>
<th>Retrofit</th>
<th>Notch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit 1</td>
<td>none</td>
<td>none</td>
</tr>
<tr>
<td>Unit 2</td>
<td>none</td>
<td>c</td>
</tr>
<tr>
<td>Unit 3</td>
<td>a</td>
<td>c</td>
</tr>
<tr>
<td>Unit 4</td>
<td>b</td>
<td>c</td>
</tr>
</tbody>
</table>

a - Full length retrofit with 3.94 in. (100 mm) wide CFRP laminate.
b - Quarter length retrofit with 3.94 in. (100 mm) wide CFRP laminate.
c - 4 in. (10.16 cm) wide notch at the mid-span of the tension (bottom) flange.

Figure 2. Lateral Supports Setup

In a typical bridge these girders would have the compression flange supported by the top slab or lateral restrained members. Because this condition would increase significantly the complexity of the test setup, a simple system of lateral supports was designed to prevent premature occurrence of lateral torsional buckling.

Four pairs of lateral brace supports were installed to reduce the predisposition for lateral torsional buckling (see figure 2.). Two pairs were installed at the support locations, and the other pairs were installed at the quarter-span location. The distance between the pairs of lateral brace system was 48 inch (121.92 cm). To prevent local buckling, web stiffeners were also installed at the locations under the load and above the reaction supports, and a loading plate was inserted between the load cell and the top flange of the specimen.

Figure 3. Position of the Strain Gages
Strain gages were installed at 4 separate sections of the girder, such as: (1) mid-span, (2) 1.5 in. (38.10 mm) from the centerline of the girder, (3) 7 in. (177.8 mm) from the centerline of the girder, and (4) 24 in. (609.6 mm) from the centerline of the girder. At each of these sections, gages were installed at several locations along the depth of the girders (see Figure 3.).

\[ e_i = 1.05 e_c \text{ at } e_{FRP}=0.8\% \]  

(7)

These expressions indicate that due to the section loss the neutral axis moves towards the compression flange and additional strains develop in the FRP laminate (see Figure 5.). So this condition indicates that the notch provided in the girder is necessary in order to properly study the influence of the retrofit of corroded steel members with FRP laminates.

4 EXPERIMENTAL RESULTS

A finite element analysis was conducted using COSMOS/M, a computer software program developed by Structural Research and Analysis Corporation (SRAC, 1994). At this stage and FEM analysis has been completed for test units 1 and 2. Load versus mid-span deflection of the FEM analysis and test results are shown in Figure 6.
The failure mode of test units 1 and unit 2 were lateral-torsional buckling, and the failure mode of test units 3 and 4 were peel off of FRP laminates. Test units 3 and 4 were loaded to failure after FRP peel off to compare the test results with test unit 2.

For the test unit 3 (full retrofit) peel off initiated at the notch (middle of the girder), and then extended to the end of the CFRP laminate as the vertical was increased. This was due to the stress concentration and high shear stresses near the notch area. For test unit 4 (14in. 355.6mm long FRP laminate retrofit scheme), the entire CFRP laminate peeled off suddenly. Failure modes of the tests are shown in Figures 10. to 12.

Comparing with the controlled notched girder (test unit 2), an increase in stiffness was achieved by the FRP retrofit scheme, as shown in Figure 7.

The maximum load capacity that test unit 3 reached was 37.2 kips (165.5kN), and the maximum load capacity that test unit 4 reached was 33.6kips. Test unit 3 had high peel off load value than test unit 4. This was because it took time for long FRP laminate to develop the full peel off. Theoretical and test results are shown in Table 3.

Table 3 Theoretical and Experimental Plastic Load Capacity

<table>
<thead>
<tr>
<th>Test unit</th>
<th>Theoretical</th>
<th>Experimental</th>
<th>Load at L/D=360</th>
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<tbody>
<tr>
<td></td>
<td>kips</td>
<td>kN</td>
<td>kips</td>
</tr>
<tr>
<td>1</td>
<td>43.5</td>
<td>193.5</td>
<td>22.1</td>
</tr>
<tr>
<td>2</td>
<td>30.5</td>
<td>135.7</td>
<td>23.9</td>
</tr>
<tr>
<td>3</td>
<td>NA</td>
<td>NA</td>
<td>37.2</td>
</tr>
<tr>
<td>4</td>
<td>NA</td>
<td>NA</td>
<td>33.6</td>
</tr>
</tbody>
</table>

L: length between supports. 
D: mid-span deflection.

The developments of peel off of the FRP laminates versus load are shown in Figures 13 and 14.

5 CONCLUSIONS

Based on the experimental results obtained under this research program, it is clear that an increase in stiffness and plastic load of corroded steel members can be achieved from the application of CFRP laminates to the tension flange of corroded steel members. In addition, preliminary analytical and experimental results indicate that notching of tension flanges to simulate the loss in steel area due to corrosion is viable for a proper study of corroded steel members retrofitted with FRP laminates.

Peel off of FRP laminates was the observed failure mode of the retrofitted girders. This was due to high stress concentrations near the girders mid-span. In order to avoid this undesirable mode of failure, in the next series of tests, a different retrofit strategy will be implemented.

In the next series of tests, GFRP sheets will be applied perpendicularly to the longitudinal laminates, which will wrap around the tension flange and part of the web. These sheets will be applied adjacent to the notched area and along the girder in order to avoid the peel off of the longitudinal FRP laminates. Future research will be
focused on; (1) strengthen the corroded steel members with hybrid FRP (CFRP & GFRP) laminates. GFRP will be inserted between CFRP and steel to prevent galvanic corrosion due to the direct contact between the CFRP laminate and steel, (2) use different surface preparation work to improve the bond strength, and (3) use GFRP sheets wrap to improve the bond strength.

6 REFERENCES


7 ACKNOWLEDGEMENTS

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