Flexural Behavior of Steel-Concrete Composite Girders Strengthened via Fiber Reinforced Plastic FRP

Abdul Qader Melhem

ABSTRACT

Many of bridge and building elements are in the need of continuous maintenance, repair, and strengthening because of the severe environmental conditions and heavy unpredicted moving loads. This paper elucidates in details theoretically and experimentally how to broaden the structural capacities of steel-concrete composite girders implementing fiber reinforced polymers FRP materials and high strength concrete HSC. Theoretical formulas derived and designing equations put forward in order to improve flexural capacities of steel-concrete composite girder models. Failure modes of those composite models will be discussed also. The experimental composite models are employed to verify the accuracy of the theoretical results. The first model is the steel beam alone, the second model is the steel-concrete composite model assembles of reinforced concrete slab connected by means of studs to the top of steel beam while the third composite model is the second composite model but fabricated with FRP plate to the bottom surface of the steel beam. The simple composite models have the same span length, steel beam section, yield and ultimate strength of steel beam and FRP thickness. High strength concrete has been utilized to improve durability. The models had been exposed to two concentrated loads increased gradually up to failure.

KEYWORDS

Composite beam, Elastic-plastic, Ultimate moment, Strengthening, Durability, Fiber reinforced polymers


1 INTRODUCTION

With increasing harsh environmental conditions (creep, cracking, de-lamination, wear, and/or effects of foreign object damage etc) along with unpredictable live loads on highway bridges and other vital structures, there is growing tendency to boost up the structural capacity of the composite components rather than replacing them according to the cost-effective standard. In recent years, there has been a renewed emphasis on improving durability and increasing service life of structures. The value of the concrete is defined in terms of maturity, permeability, air-void structure quantification, sulfate resistance, chloride penetration, strength, and in situ performance [David and Meyerhof 1958].

Strengthening of existing steel-concrete composite structures is becoming a major concern for the industries due to age, steady increase in the applied loads, environmental effects, lack of adequate maintenance etc. The advent of fiber reinforced polymers FRP has been explored as new materials for strengthening of existing structures. These innovative materials are characterized by their high strength to weight ratio and environmental and fatigue sturdiness. It has been employed successfully in the reinforced concrete structures at least for the last thirty years, but until recently has been utilized momentarily in the composite structures. In fact, the FRP plates, laminates or sheets can be epoxy bonded to the tension face of the composite members to enhance their strength and stiffness.

An earlier study conducted, at the University of South Florida, using CFRP in the repair of steel-concrete composite bridges [Sen and Liby 1994]. Other researchers have investigated the possibility of strengthening steel beams with CFRP plates [Mertz and Gillespie 1996]. The effectiveness of using FRP to increase the stiffness of steel–concrete composite beams also has been demonstrated experimentally [Tavakkolizadeh and Saadatmanesh 2003 and Al-Saidy et al 2004]. Using of externally bonded high modulus carbon fibre reinforced polymer (HM CFRP) materials to strengthen steel bridges and structures has been proposed along with guidelines and installation techniques based on the best practice reported in the literature and the extensive practical experience in bonding of composite materials [Schnerch et al 2007].

2 BASIC ASSUMPTIONS AND OBJECTIVES

The impending failure manners for a steel-concrete composite beam strengthened with FRP materials are: rupture of the FRP plate, de-bonding of the FRP plate, and crushing of the compression concrete due to loading, environmental conditions or both. There are other failure modes such as, buckling of the compression flange and shear failure of the web. To prevent de-bonding of the FRP plate, it is important to give a ruling for the actual state of stress near the end of the plate. The main objectives of this research are:
- To strengthen previously damaged or distressed steel-concrete composite members making use of FRP plates.
- To build up a procedure that could be used for strengthening of existing steel-concrete composite members, using FRP plates, subjected to up-normal loads or to severe atmospheric conditions.
- To design steel-concrete composite members making use of FRP plates for architectural purpose, such as: reducing the height, weight, etc.

3 STRESSES OWING TO ENVIRONMENTAL CIRCUMSTANCES

These stresses are by reason of creep, shrinkage, relative temperature differences, difference between thermal expansion of concrete and steel, and concrete growth. During a sudden rise in temperature, the steel beam and probably the FRP become warmer than that the connected concrete slab because of quicker temperature stabilization of steel. This kind of temperature differential creates an additional strain of $1x(10)^4$ which should be considered in the design. The net combination effect of creep, shrinkage, and temperature differential between cold concrete and warmer steel is shown in Fig. T 11, Flexural Behavior of Steel Concrete Composite Girders Strengthened via Fiber Reinforced Plastic FRP, Abdul Qader Melhem
The effect of difference between thermal expansion of concrete and steel is shown in Fig. 1.b. Concrete growth causes stresses in composite section similar to that produced by differences in thermal expansion coefficients of steel and concrete. Concrete growth may be due to physical causes such as; freezing-thawing, wetting-drying, heating-cooling, etc. The chemical cause are due to certain cement-aggregate combinations.

4 FULL PLASTIC BENDING

When the applied moment becomes greater than that moment which generates first yield of farthest fibers, portions of the composite section will keep hold of a constant yield stress, at strains which go beyond the elastic limit. Progressive straining and yielding of the material will then take place until complete plasticization of the section. At this bound no additional moment can be developed, thus a full plastic moment is reached. At this stage, the plastic neutral axis may be located in the concrete slab, steel top flange, or steel web.

The failure load with shoring is almost the same as that developed when no shoring is used. This is because of redistribution of stresses in the steel beam, which has little effect on the final plastic moment capacity of the beam. The amount of longitudinal shear which must be taken in to account should include that produced by the dead loads (if shoring is used), live loads, creep, shrinkage, temperature difference, and any difference between the thermal coefficients of expansion of slab and steel beam. From previous numerous tests, it has been shown that if shear connectors are properly designed, composite beam failure is due to only ultimate compressive stress in the concrete slab or full plasticity in the steel beam, or both combination.

4.1 Neutral Axis in the Concrete Slab (Fig. 2a):

If: $0.85 f'_c A_c > A_y F_y + A_{fp} f_{y,fp}$, the plastic neutral axis is located within the concrete slab. The plastic neutral axis location and ultimate moment are given by these equations:

$$a_{T1} = \frac{A_y F_y + A_{fp} f_{y,fp}}{0.85 f'_c b_e}$$

(1)

$$M_u = 0.85 f'_c b_e a_{T1} [d + t_c - d_T - 0.5 a_{T1}] + T_{fp} (d_T + 0.5 t_{fp})$$

(2)

Where,

- $T_s = A_s F_y$
- $T_{fp} = A_{fp} f_{y,fp}$

$$d_T = (1/A_s)[A_{bf} (0.5 t_{bf}) + A_w (t_{bf} + \frac{d_w}{2}) + A_{bf} (t_{bf} + d_w + \frac{t_{bf}}{2})]$$

When the section is symmetric: $d_T = d/2$
As = Area of steel section = (Abf + Aw + Atf)
Afrp = bfrp tfrp = Area of FRP plate
Fy = Yield stress limit of steel beam, f_{y,frp} = Ultimate tensile strength of the FRP material
Other terms are shown in Fig. 2

4.3 Neutral Axis in the Steel Section (Fig. 2b, 2c):

If: $0.85 f'_c A_c < F_y A_s + A_{frp} f_{y,frp}$, the plastic neutral axis is located in the steel section.

1. If: $F_y A_{sf} > C^T$, the plastic neutral axis is in the top flange of the steel section.
   Where,
   
   \[
   C^T = 0.5 \left[ (F_y A_s + A_{frp} f_{y,frp}) - 0.85 f'_c A_c \right]
   \]

The location of neutral axis and ultimate bending moment are given by these equations:

**Figure 2.** Plastic analysis of steel-concrete composite model.
$$a_{T2} = \frac{C^T}{F_y b_T}$$  \hspace{1cm} (4)

$$M_u = 0.85 f'_c b_T t_c \left[ d + 0.5t_c - d_T \right] + C^T(d - d_T - 0.5 a_{T2}) + T_{fp} \left( d_T + 0.5 t_{fp} \right)$$  \hspace{1cm} (5)

$$d_T = \left( \frac{1}{A_{w1}} \right) [A_{bf} (0.5 t_{bf}) + A_w (t_{bf} + \frac{d_w}{2}) + b_{tf} (t_{bf} - a_{T2}) (t_{bf} + d_w + \frac{t_{frp} - a_{T2}}{2})]$$

$$A_{w1} = A_{bf} + A_w + b_{tf} (t_{bf} - a_{T2})$$

2. If: \( F_y A_{tf} < C^T \), the plastic neutral axis is in the web of the steel section.

The location of neutral axis and ultimate bending moment are given by these equations:

$$a_{T3} = \frac{C^T - A_{y} F_y}{F_y t_w}$$  \hspace{1cm} (6)

$$M_u = 0.85 f'_c b_T t_c \left[ d + 0.5t_c - d_T \right] + C^T[d - (t_{bf} + a_{T3}) + a_{T4} - d_T] + T_{fp} (d_T + 0.5 t_{fp})$$  \hspace{1cm} (7)

$$d_T = \left( \frac{1}{A_{w2}} \right) [A_{bf} (0.5 t_{bf}) + t_w (d_w - a_{T3}) (t_{bf} + \frac{d_w - a_{T3}}{2})]$$

$$A_{w2} = A_{bf} + t_w (d_w - a_{T3})$$

## 5 SPECIMEN DETAILS

A steel-concrete composite girder model has been reproduced and reworked again in more specifics [Sen, et al, 2001] using high strength concrete (HSC) to boost up the durability. This is another way to heighten service life through designing, specifying, and building structures using (HSC). The American Concrete Institute has previously defined (HSC) as concrete with a compressive strength greater than 41 MPa (6000 Psi).

The aim of the test is to evaluate the viability of using FRP plates for improving the strength of steel-concrete composite girder models. Table 1 present material properties of the models. Figure 3 shows cross section of the model. The simple model was subjected to two concentrated loads as been show. The loads were applied through two 710 mm transverse stiffened spreader beams welded to a 1220 mm long longitudinal stiffened beam. The loads were two uniform transverse line loads spaced 1220 mm apart, transferred equally to the two spreader beams.

### Table 1. Properties of Materials.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Steel</th>
<th>FRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength, MPa</td>
<td>43.6</td>
<td>370</td>
</tr>
<tr>
<td>Elastic Modulus, MPa</td>
<td>13800</td>
<td>198300</td>
</tr>
<tr>
<td>Peak strain</td>
<td>0.00197</td>
<td>0.31</td>
</tr>
<tr>
<td>Failure strain</td>
<td>0.00380</td>
<td>Poisson’s ratio</td>
</tr>
</tbody>
</table>

The cross section consists of a 710 mm reinforced concrete slab width, 114 mm thickness attached to a W 8 x 24 steel section via 36 stud shear connectors. The studs were spaced at: @ 127.6 mm for the shear spans and @ 254 mm for the distance between concentrated loads. The FRP plate length was 3650 mm, bonded to the bottom steel surface using epoxy adhesive. Table 1 presents mechanical properties of the model materials. The first phase of the experimental program was testing W 8 x 24 steel section under two concentrating loads. The second phase of the experimental program was the steel-concrete composite girder model had been loaded beyond yield stress limit of its bottom tension flange, which is 0.55 \( F_y \), according to the AASHTO Bridge Specifications.

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As a result of this loading, higher than yield stress limit of tension flange, there was permanent deformation prior to strengthening by means of FRP plate. The third phase of the experimental program was the deformed steel-concrete composite girder model had been strengthened with 2 mm thick, 165 mm width and 3650 mm length epoxy adhesive. After the adhesive cured for 48 h, the ends of FRP plate were bolted to resist the peeling stresses. The same loading scheme had been used for testing the repaired model. The test was stopped when no extra load could be applied and the deflection exceeded the 100 mm limit. No adhesive failure had been observed. Table 2 presents elastic model properties. These properties have been calculated using AASHTO Bridge Specifications.

Table 2. Elastic properties of the sections.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$y_{bot}$ (mm)</th>
<th>$y_{top}$ (mm)</th>
<th>$I_{com}$ (cm$^4$)</th>
<th>$S_{bot}$ (cm$^3$)</th>
<th>$S_{top}$ (cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite beam</td>
<td>216.8</td>
<td>98.6</td>
<td>13177.11</td>
<td>607.77</td>
<td>1336.28</td>
</tr>
<tr>
<td>Composite beam with FRP</td>
<td>216.5</td>
<td>101.0</td>
<td>14072.48</td>
<td>650.15</td>
<td>1393.73</td>
</tr>
</tbody>
</table>

The deflection at midspan is calculated by this equation:

$$\Delta_{L/2} = \frac{(P/2)}{6EI_v} \left(\frac{a}{L}\right) \left[ \frac{3}{4} \left(\frac{a}{L}\right)^2 \right] \approx \frac{1}{51} \frac{PL^3}{EI_v}$$ (8)

The strains at the top and bottom of the steel-concrete composite section are given by these equations:

$$\varepsilon_t = \frac{Pa}{2EI_v} y_{top}$$ and $$\varepsilon_s = \frac{Pa}{2EI_v} y_{bot}$$ (9)

Where,
L = Span length between supports, equal to 5948 mm  
a = Shear span, the distance between support and first load which is equal to 2364 mm  
a/L = Shear span ratio is equal to 0.3974  
E = Modulus of elasticity of the homogeneous section  
Itr = Moment of inertia of composite section  
P = Applied load  
y_{top} and y_{bot} = Distances from neutral axis to the top and bottom of the steel-concrete composite section

Table 3 presents summary of the theoretical and experimental studies. Figure 4 shows three load-displacement curves; the top one is for the composite girder strengthened with FRP plate, the middle is for the composite girder without FRP plate and the lower curve is for the steel section alone.

Table 3. Summary of outcomes.

<table>
<thead>
<tr>
<th>Beam</th>
<th>a_{21} (mm)</th>
<th>P_{th} (KN)</th>
<th>P_{ult} (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Thoe.</td>
<td>Exp.</td>
</tr>
<tr>
<td>Composite beam</td>
<td>53.8</td>
<td>125</td>
<td>130</td>
</tr>
<tr>
<td>Composite beam with FRP</td>
<td>87.3</td>
<td>131</td>
<td>150</td>
</tr>
</tbody>
</table>

These curves outline the phases of the experimental program. The curve for repaired or strengthened composite beam is gaining more strength in the post – yield region as been shown. The simulating service damage load for un-strengthened composite beam was 186 KN, while the ultimate load for strengthened composite beam was 271, a strength gain of 46 %. On the other hand, the predicted ultimate load for unstrengthened composite beam was 249 KN while the ultimate load for strengthened composite beam 271 KN, a strength gain of 8.8 %. This low value is attributed to the fact that the FRP plate has a lower elastic modulus than that of steel; 198300 MPa versus 114000 MPa. So, the stresses in FRP plate were less than those stresses in the bottom steel flange prior to yielding.

Figure 4. Load displacement curves for the steel-concrete composite girder models.

6 DESIGNING EQUATIONS

Designing equations have focused on the case where the neutral axis is within the slab (Fig. 2a). The Majority of steel-concrete composite girders full under this category. The designing equations may be summarized as followings (Eq. 1 and Eq. 2):
\[ q_{frp}^2 + 2 \left[ q_s - \frac{d}{t_c} - 1 \right] q_{frp} + 2 \left[ 0.5q_s - \left( \frac{d - d_T}{t_c} \right) - 1 \right] q_s + \frac{M_u}{0.85f'c_b t_c^2} = 0 \] (10)

\[ q_{frp} = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \]

Where: \( a = 1 \), \( b = 2 \left[ q_s - \frac{d}{t_c} - 1 \right] \), \( c = 2 \left[ 0.5q_s - \left( \frac{d - d_T}{t_c} \right) - 1 \right] q_s + \frac{M_u}{0.85f'c_b t_c^2} \)

\[ q_{frp} = \frac{A_{frp}}{A_c} \frac{f_{frp}}{0.85f'_c} \] and \( q_s = \frac{A_s}{A_c} \frac{F_y}{0.85f'_c} \)

7 CONCLUSIONS

- The presence of FRP plate increases the stiffness of the steel-concrete composite model provided that adequate anchorage of the FRP plate to the bottom steel flange. Therefore, the composite beams strengthened with FRP plates using HSC can be utilized effectively in durability practice.

- A ductile failure mechanism comes about for the repaired or strengthened composite model with FRP reinforcement accompanied with considerable deformation, provided that shear connectors are properly designed against heavier loads and environmental conditions.

- The deformed or distressed sections can be strengthened successfully via FRP plates a substantial strength gain was attained.

- In addition to increases in ultimate load, the elastic region of the un-strengthened composite beam was significantly extended in the repaired composite beam. This provides a valuable marginal safety factor.

- The designed equations can be employed for selection of the area fraction of FRP plate needed to upgrade existing bridges superstructures, provided that ultimate bending moment is available.

REFERENCES


