NUMERICAL ANALYSIS OF FRP PEDESTRIAN BRIDGE IN TAIJIANG NATIONAL PARK

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ABSTRACT

In this paper, the numerical analysis of the pedestrian bridge made of glass fiber reinforced plastic (GFRP) components in Taijiang National Park, Tainan, Taiwan was introduced. The bridge’s superstructure used 4 GFRP I-girders which are connected to each other with GFRP plate decks and GFRP stiffeners were used to resist shear while also helping transmit forces to adjacent girders. This study aims to analyze the bridge superstructure by modeling it using ANSYS in the finite-element analysis. Also, small scale analysis of a 6-m long and an 8-m long girder were performed. The finite-element results were compared with the Euler and Timoshenko beam theoretical values.

KEYWORDS

FRP, GFRP, Girder-deck system, finite-element analysis, pedestrian bridge, stiffener.

INTRODUCTION

The biggest problem facing infrastructure structures built in Taijiang National Park is corrosion. A visit to the site revealed that most of the exposed steel members such as girders and railings have been partly or greatly corroded. The high presence of chloride ions from the surrounding seawater has worsened the deterioration phenomenon of existing structures in the park. This makes the lifecycle cost of those structures due to maintenance far higher than the initial construction cost. Therefore, this makes high-strength and corrosion-resistant fiber reinforced plastic (FRP) composite materials more than viable for a pedestrian bridge built in the park. This 8-m long
pedestrian bridge is the first of its kind in Taiwan to be composed exclusively of FRP components only. In this paper, the bridge and its main structural components are modeled and analyzed using ANSYS finite-element program. Similar works and related literatures on the numerical analysis of FRP pedestrian bridges and bridge components are presented in the following review.

LITERATURE REVIEW

Since Taiwan isn’t the first in the world to use FRP bridges, a few literatures from abroad are presented here. Salim et al. (1997) introduced a design concept for a short-span FRP bridge composed of cellular FRP deck and an optimized FRP winged-box beam as the stringers. Based on the present first-order SDMF orthotropic plate solution, a simplified design analysis procedure is proposed, and it can be used for the analysis and design optimization of various case studies of FRP deck-and-stringer bridges. Qiao et al. (2000) proposed a systematic approach for design analysis of FRP deck/stringer bridge systems. The design approach includes the analyses of ply (micro-mechanics), panel (macro-mechanics), beam or stringer (mechanics of laminated beam), deck (elastic equivalence model), and combined deck/stringer system (series approximation technique). Schniepp et al. (2002) tested double-wed FRP beams of different spans and depth in three-point bending and adopted the Timoshenko Beam Theory to calculate the shear stiffness for a span-to-depth ratio \( (L/d) \) of 6–10 as this is significant in predicting the failure mode of the beam. Hejll et al. (2005) use large scale hybrid FRP composite girders in a bridge structure. The manufacturing process, theory and field application of the girders was also carried out. Finite-element results were compared with theoretical and experimental results. Mendes et al. (2011) designed a single 12-m span I-section footbridge affixed a layer of carbon fiber patch at the top flange and analyzed the mechanical behavior of the structure by the finite element method. From the numerical and experimental research, it was concluded that the connection between the deck and the profile can be made exclusively of epoxy resin. It was also concluded that the bridge design is conditioned by service limit state due to the deflection.

STRUCTURAL DESIGN OF THE FRP PEDESTRIAN BRIDGE

FRP composites is one of the materials being researched and tested in an attempt to make a new structural material that is superior to reinforced concrete and steel for specific applications. The former two traditional building materials prove to be incapable to providing the corrosion resistance, better fatigue, low density and high strength-to-weight ratio enjoyed by FRP components. The superstructure of the FRP pedestrian bridge uses GFRP components bonded to each other either adhesively using epoxy resin or by using GFRP bolts in addition to the adhesive bonding. No metallic material at all was used for the pedestrian bridge. The following describes the structural design of each item employed in the superstructure of the bridge:

**Girder:** The main girder is designed using four 20 cm wide, 41 cm high, and 800 cm long (with span length of 750 cm) single-web I-girders. In the configuration of the main girder of the bridge, stiffeners (made of FRP end plates) are placed in-between the four I-girders at \( L/4 \), \( L/2 \) and \( L/4 \) of the I-girder with the intention of resisting
the shear force and transmitting the applied force to the adjacent girders. In addition, FRP rods bolted at midpoint of the partitioned stiffeners would help resist against torsion at the ends and midpoint of the pultruded girders. This thus enhances the performance of the girders and the mechanical behavior of the whole girder-deck system and prevents failure of a single girder of the system caused by irresistible external force by reversing the failure. The design of girder and stiffeners of the FRP Pedestrian Bridge at Taijiang National Park are shown in Figure 1.

Figure 1. Design of girder and stiffeners of the FRP pedestrian bridge at Taijiang National Park

Bridge deck: 16 GFRP composite plates, each measuring 150 cm long, 50 cm wide and 1.2 cm thick, were used as the bridge deck. These are bonded physically and chemically to the girders of the stiffeners by GFRP pins and adhesive resin respectively. After completion of assembly of the FRP girder-deck system, a light crane was used to hoist the superstructure of the bridge into position on the abutments.

Deflection Design Requirement

For FRP composite bridges, the specified deflection values are more liberal due to the high strength, but low stiffness (modulus of elasticity) characteristics of the material. Because of the low modulus, GFRP composite bridges tend to be at very low levels of stress (in comparison to other materials) at a certain deflection limit. To allow better use of the FRP materials while maintaining a high factor of safety, Table 1 gives famous design specifications considered in this paper.

Mended et al. (2011) mentioned that the design of an FRP bridge is conditioned by service limit state due to the deflection. With such little stiffness of the GFRP profiles, deflection control design was employed for the Taijiang National Park FRP Pedestrian Bridge. In this study, therefore, the maximum allowable deflection of L/300 of the DMF RB Vol. 1, Sec. 3, Part 17 is chosen.
Table 1. FRP bridge deflection requirement

<table>
<thead>
<tr>
<th>Nation</th>
<th>Specification Name</th>
<th>Scope of Application</th>
<th>Design Method</th>
<th>Deflection Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.K.</td>
<td>DMFRB Vol. 1, Sec. 3, Part 17</td>
<td>FRP highway bridges</td>
<td>Ultimate Limit State</td>
<td>$L/300$</td>
</tr>
<tr>
<td>U.S.A.</td>
<td>A Guide to Fiber-Reinforced Polymer Trail Bridge</td>
<td>FRP Trail bridges</td>
<td>Working Stress Method</td>
<td>$L/400$</td>
</tr>
<tr>
<td></td>
<td>Guide Specifications for Design of FRP Pedestrian Bridges</td>
<td>FRP Pedestrian Bridges</td>
<td>LRFD</td>
<td>$L/500$</td>
</tr>
</tbody>
</table>

**NUMERICAL ANALYSIS**

The Euler-Bernoulli Beam Theory (EBT) and Timoshenko Beam Theory (TBT) were employed to verify the Finite Element Analysis (FEA) of the pultruded GFRP girders. After the verification, the finite element model of the whole superstructure of the bridge was made. The following gives details of both the theoretical and numerical analysis.

**Timoshenko Beam Theory**

The choice of beam theory depends on a lot of factors, one of which is the degree of anisotropy of the composite material (Neto and Rovere 2007). Composite materials display in general a longitudinal-to-shear modulus ratio considerably higher than those found in isotropic materials and this ratio tends to increase as the anisotropy degree of the material increases. For this reason shear deformation in the beam will increase as the anisotropy degree of the material increases. To account for shear deformation, TBT gives a better approximation of the actual beam as compared to the traditional EBT. In the TBT, the plane sections of the girder are assumed to remain plane, but no longer normal to the beam’s neutral axis (Neto and Rovere 2007). In the present paper, with a relatively small aspect ratio (aspect ratio corresponds to length-to-height ratio) of the GFRP profiles, shear deformations needs to be taken into consideration. The popular Timoshenko beam formula for the maximum deflection of a simply supported beam under uniformly distributed load is given as follows:

$$\delta_{\text{max}} = \frac{5wL^4}{384EI} + \frac{wL^2}{8GA}$$

(1)

Where $w$ is the intensity of the distributed load; $L$ is the span or the distance between supports; $A$ is the section area and $I$ is the section moment of inertia with respect to the x-axis; $E$ is the flexural elastic modulus or Young’s modulus for isotropic materials; $G$ is the shear modulus and $\kappa$ is the shear coefficient to account for the fact that shear stress distribution is non-uniform across the section. For an I-section girder, the shear coefficient is given by the following relation in which $\nu$ is the Poisson’s ratio:

$$\kappa = \frac{10(1 + \nu)(1 + 3\nu)^2}{(12 + 72m^2 + 90m^3) + (11 + 66m + 135m^2 + 90m^3) + 30n^2(m + m^2) + 5m^2(8m + 9m^2)} = 0.682$$
On the other hand, Ghugal and Sharma (2011) found for a simply supported beam with a uniformly distributed load, the maximum deflection is given as by:

\[
\delta_{\text{max}} = \frac{5wL^4}{384EI}\left[1 + \frac{1}{2}(1 + \nu)^3\right]
\]

Here, \(w\) is the intensity of the transverse load; \(L\) is the span or the distance between supports; \(I\) is the section moment of inertia with respect to the x-axis; \(E\) is the flexural elastic modulus or Young’s modulus for isotropic materials; \(\nu\) as the Poisson’s ratio of the girder; \(h\) is the height of the girder section. These two equations will be used for the TBT.

**Euler-Bernoulli Beam Theory**

The Euler-Bernoulli Beam Theory can be considered as a special case of the Timoshenko Beam theory in which the shear effect is ignored for long beams. It is a simplification of the linear theory of elasticity which provides a means of calculating the load-carry capacity and deflection of beams. For our case of a simply supported girder with a uniformly distributed load on the top longitudinal span of the GFRP girders, the Euler-Bernoulli equation is given for the maximum deflection as:

\[
\delta_{\text{max}} = \frac{5wL^4}{384EI}
\]

where \(w\) is the intensity of the transverse load; \(L\) is the span or the distance between supports; \(I\) is the section moment of inertia with respect to the x-axis; \(E\) is the flexural elastic modulus for isotropic materials.

**Finite Element Analysis**

For the finite element analysis (FEA) of the bridge girder, the general purpose commercial finite element analysis software, ANSYS, was used for the static linear analysis. The bridge girders use an 8-node Solid 45 element with three degrees of freedom for each element in the Finite element mesh (Figure 2). Table 2 shows the mesh details. Boundary conditions or constraints were applied as structural displacement loads within ANSYS. A simply support beam condition was used to simulate the girder with a vertical displacement (in the \(y\) direction) and transverse displacement (\(x\) direction) restrain on the nodes of both ends of the profile bottom flange. An extra constraint in the \(z\) direction was made at one of the ends of the girder. The constraints as applied in the above simply supported beam are also extended to all the girders in the bridge girder-deck system.

<table>
<thead>
<tr>
<th>Table 2. Mesh details of the pedestrian bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck</td>
</tr>
<tr>
<td>No. of volumes meshed</td>
</tr>
<tr>
<td>Max. node number</td>
</tr>
<tr>
<td>Max. element number</td>
</tr>
</tbody>
</table>

The effects of self-weight of all the structural elements are considered by assigning a density of 1800 kg/m³ to all
the material models. An acceleration of 9.81 \( m/s^2 \) due to gravity is then applied as a global inertia load. The girder-deck system had a surface load of 40 \( MPa \) applied on the whole area of the bridge deck in agreement with the load design requirements. Consequently, the bridge’s superstructure was modeled with the four GFRP I-girders as the diaphragm using SOLID45 element. The stiffeners and deck were also modeled but not the FRP rod shown in Fig. 2(b) and Fig. 2(a), respectively. Constraints were applied at both ends in a simply supported condition before applying a vertical surface load of 40 \( MPa \). Material properties of GFRP components listed in Table 3 below were used in the Finite element analysis. Special attention was given to the use of consistent sign convention throughout the whole analysis to ensure that the correct Young’s Modulus, shear modulus and Poisson’s ratio values are assigned. The analysis results converged well and Table 2 shows the material properties of the GFRP profiles used in this study.

![finite element mesh of bridge superstructure](image)

(a) Whole bridge superstructure  (b) Deck  (c) Stiffeners

Figure 2. Finite element mesh of bridge superstructure

<table>
<thead>
<tr>
<th>Property</th>
<th>( E_x ) (MPa)</th>
<th>( E_y ) (MPa)</th>
<th>( E_z ) (MPa)</th>
<th>( G_{xy} ) (MPa)</th>
<th>( G_{yz} ) (MPa)</th>
<th>( G_{xz} ) (MPa)</th>
<th>( v_{xy} )</th>
<th>( v_{yz} )</th>
<th>( v_{xz} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>All girders</td>
<td>6621</td>
<td>6621</td>
<td>20690</td>
<td>3724</td>
<td>3724</td>
<td>3724</td>
<td>0.33</td>
<td>0.11</td>
<td>0.11</td>
</tr>
<tr>
<td>Deck plate</td>
<td>6621</td>
<td>20690</td>
<td>6621</td>
<td>3724</td>
<td>3724</td>
<td>3724</td>
<td>0.11</td>
<td>0.11</td>
<td>0.33</td>
</tr>
<tr>
<td>Stiffeners</td>
<td>20690</td>
<td>6621</td>
<td>6621</td>
<td>3724</td>
<td>3724</td>
<td>3724</td>
<td>0.11</td>
<td>0.33</td>
<td>0.11</td>
</tr>
</tbody>
</table>

\( ^a_X = \) transverse (perpendicular to traffic), \( Y = \) vertical direction and \( Z = \) longitudinal (traffic) direction

**Constitutive Equation of FRP Girder**

In the constitutive model, the GFRP material was considered orthotropic and linearly elastic. The constitutive relations for orthotropic FRP materials used in the finite-element analysis are:
where,

\[ \Delta = 1 - \frac{V_{xy} V_{yz} - V_{yx} V_{zy} - V_{zx} V_{xz} - 2V_{yz} V_{zy} V_{xz}}{E_y} \]  

(4b)

\[ \frac{V_{zx}}{E_x} = \frac{V_{xy}}{E_y}, \quad \frac{V_{xy}}{E_y} = \frac{V_{yz}}{E_z}, \quad \frac{V_{yz}}{E_z} = \frac{V_{zx}}{E_x} \]  

(4c)

In the above equations, \( \sigma \) represents the normal stresses and \( \gamma \) is the shear stresses in the directions specified in the note below Table 2. In the same way, \( E \), \( G \) and \( \nu \) follow the directions noted in Table 2.

**COMPARISON OF THE ANALYTICAL RESULTS**

The theoretical analysis uses an equivalent Young’s modulus \( E = 1,7161 \text{ MPa} \) given by the manufacturer of the GFRP profiles and a Poisson’s ratio, \( \nu \) of 0.33. The value of the uniformly distributed load, \( w \) is obtained by multiplying the design load of 40 MPa by the width (0.53 m) of load to be taken by an interior girder. This produced a uniformly distributed line load of 2.185 kN/m after adding a dead load (a density of 1,800 kg/m\(^3\)) due to the self-weight of the girder.

**Table 4. Results comparison**

<table>
<thead>
<tr>
<th>Analytical methods</th>
<th>6-m girder</th>
<th>8-m girder</th>
<th>Girder-deck system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Deflection (cm)</td>
<td>Deflection (cm)</td>
<td>Deflection (cm)</td>
</tr>
<tr>
<td>FEA</td>
<td>0.52</td>
<td>1.38</td>
<td>1.06</td>
</tr>
<tr>
<td>EBT</td>
<td>0.493</td>
<td>1.379</td>
<td></td>
</tr>
<tr>
<td>Ghugal and Sharma</td>
<td>0.499</td>
<td>1.389</td>
<td></td>
</tr>
<tr>
<td>Equation (1)</td>
<td>0.518</td>
<td>1.419</td>
<td></td>
</tr>
<tr>
<td>Safety Factor</td>
<td>36</td>
<td>41</td>
<td></td>
</tr>
</tbody>
</table>

It is clearly seen in Table 4 that the results are closely the same. The 6-m girder has more close deflection values in the TBT than in the EBT when compared to the FEA for verification. This could be due to the effect of shear deformation considered for the girder. On the other hand, there is more correlation in the deflection values of the 8-m girder with that of the FEA as the effects of shearing seems to be less as the length increases for the same section. The FEA output results for the 6-m, 8-m and the girder-deck system are shown in Figure 3 below.
CONCLUSION

A pedestrian bridge was built for the first time in Taiwan using FRP composite materials only. The superstructure of the bridge was designed using the four FRP stringers and also included stiffeners and an FRP rod. A numerical model of the bridge is obtained using ANSYS. First, a shorter girder was analyzed using the finite element software and compared with the theoretical results. Next, a girder with the same length as the bridge was analyzed numerically and results were compared with those of the theoretical results. Both results of the TBT and that of the EBT closely agree with the FEM results. Furthermore, the girder-deck system with a maximum deflection value of only 1.06 cm meets the design requirement of a limit deflection of L/300 with reasonable safety factors. Therefore, it can be concluded that the design of the bridge pedestrian bridge at Taijiang National Park is accurate and that it will provide reasonable safety for its users.

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