Field Performance of a New Fiber-Reinforced Polymer Deck

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Abstract: The field performance of a new bridge deck system composed of pultruded trapezoidal fiber-reinforced polymer (FRP) tubes wrapped in an outer wrap is described in this paper. The proof of concept FRP deck was installed on a rehabilitated bridge in Bolivar, New York, in 2012 as part of the Highways for LIFE Program from the Federal Highway Administration (FHWA). Based on the field test, the writers developed a three-dimensional finite-element model of the bridge superstructure to further study the interaction of the FRP deck and girders. Experimental and analytical data was used to determine that no composite action was developed by the new deck-girder connection. Comparisons between distribution factors based on field measurements and standard specifications show that the lever rule should be used to estimate live load distribution in similar bridge girder spacing. The serviceability performance of the bridge was found adequate; girder deflections met standard limits, and while the FRP deck experienced deflections larger than typically expected, they were confined to the localized area of the wheel load and would not affect bridge users. DOI: 10.1061/(ASCE)CF.1943-5509.0000666. © 2014 American Society of Civil Engineers.

Author keywords: Bridge deck; Composite materials; Deflection; Design; Distribution factors; Fiber-reinforced polymer; Finite-element model; Rehabilitation.

Introduction

About 25% of the United States’ 600,000 bridges are classified as structurally deficient or functionally obsolete (ASCE 2013). These bridges require rehabilitation to repair decaying members or meet new design standards. Specifically, there is a need to rehabilitate bridges with decks at the end of their design life or that have deteriorated from environmental effects and road salts. Fiber-reinforced polymer (FRP) bridge decks provide a fatigue and corrosion resistant alternative to traditional deck replacement materials while providing a solid surface that protects the structural members. Lightweight FRP decks can be installed on load posted bridges, reducing the dead load, and therefore increasing live load carrying capacity. The prefabricated decks require reduced installation time, reducing road closings and detours. Due to high initial material costs and lack of experience with FRP decks, designers often revert to traditional concrete or timber decks even though their lifetime costs associated with the required maintenance may be higher.

The Federal Highway Administration (FHWA) Highways for LIFE Technology Partnerships program joined with Bridge Composites LLC to develop, produce, and test a new economical FRP deck that can be routinely used for rehabilitation. The writers were tasked with refining existing materials, practices, and construction details to limit costs and to verify the results through a proof of concept installation on a steel girder bridge (O’Connor 2013b). The new deck was installed on the rehabilitated Pleasant Street Bridge in Bolivar, New York, in August 2012; the work was done using the Allegany County Department of Public Works (DPW) maintenance crew.

Based on the field test conducted as part of a FHWA Highways for LIFE project, the writers developed a three-dimensional (3D) finite-element (FE) model of the bridge superstructure to further study the interaction of the FRP deck and girders. Experimental and analytical data is used in this paper to determine the level of composite action developed by the new deck-girder connection, establish distribution factors for the steel girders, and observe the serviceability performance of the bridge. Results are compared with the AASHTO (2012) Load and Resistance Factor Design (LRFD) specifications and recommendations are provided.

Deck Development and Testing

The Pleasant Street Bridge required a solid surface deck with approximately the same weight as an open steel grate deck to eliminate runoff onto the girders without increasing the dead loads (O’Connor 2013b). Through the Highways for LIFE project, the writers developed a new bridge deck system composed of pultruded trapezoidal FRP tubes wrapped in an outer layer (Fig. 1) to meet these requirements. The design was optimized through 3D FE modeling conducted at the University at Buffalo.

The basic components of the deck panels, and tubes were produced by an FRP manufacturer using the pultrusion process. A fire-resistant vinyl ester resin was used as the matrix. The tube reinforcement is comprised of E-glass rovings, woven mat, and chopped strand mat. Details of the fiber layout of the pultruded tube
flanges and webs can be found in O’Connor (2013b). The tubes were assembled into panels by adhesively bonding them with epoxy and wrapping the resulting panel with additional fabrics. The vacuum-assisted resin transfer molding method was used to create an integral deck with the outer wrap. The wrap consists of layers of stitch-bonded biaxial fabrics and a continuous filament mat (randomly oriented fibers). Panel cross section dimensions are provided (Fig. 1). Laboratory testing at Pennsylvania State University verified the performance of structural panels in flexure, shear, and fatigue. The joints between panels, wearing surface, and the bridge railing post to panel connection were also experimentally verified at Pennsylvania State University before the proof of concept installation (O’Connor 2013a).

The Pleasant Street Bridge in Bolivar, New York, is 12.2-m long by 6.8-m wide carrying two lanes with 1 W530X101 girders at 0.61-m spacing. The original year 1965 open steel grate deck allowed water and road salts to heavily corrode the support structure and warranted replacement (Vanderbrook and Lehman 2011). The Allegany County DPW replaced the existing guardrails, backwalls, three of the 11 steel girders, and the entire deck in August 2012. The bridge required a lightweight solid surface deck to eliminate roadway drainage on the girders and maintain approximately the same dead load as the previous open steel grate deck. Twelve prefabricated FRP panels were manufactured to meet these requirements. Although their initial material cost was higher than conventional alternatives, the project team agreed that the most important design criterion for this project was deck performance (O’Connor 2013b).

The Allegany County DPW is familiar with installing glulam decks over steel girders. In addition, the writers’ experience with other types of FRP deck systems led to the decision that the current deck should have no penetrations from the top. Thus, an expansion hollo-bolt and stainless steel clip connection was used (Fig. 2), similar to the lag bolt connection studied by Kimmel et al. (1999) for glulam decking. Prefabricated hardwood shims created a 2% normal grade. Once the shims and panels were in place, holes were drilled into bottom of the FRP deck through predrilled holes in the shims. This installation step proved to be slow and cumbersome, future deck installations might benefit from predrilled holes in the FRP deck. After drilling was completed, the bolted connections were then installed. Shear studs, installed on the top flange of each beam at the midspan panel joint, created a fixed connection along the entire width of the deck. An epoxy grout was used to fill the voids between all transverse and longitudinal panel joints to create deck continuity and eliminate drainage onto the steel girders. Finally, the DPW applied the second course of the wearing surface coat and installed the guardrails by bolting them to the deck.

Live load testing was completed in November 2012 with two loaded 320-kN, triaxle trucks with their tag axles lifted. The trucks passed over four transverse load paths (Fig. 3). Load Path A positioned the truck over the bridge centerline. Load Path B placed the outer wheel over girder 1 while Load Path C ran the outer wheel line between girders 1 and 2. Paths BN and CN are symmetric to B and C about the centerline. Both trucks repeated Load Paths C and CN simultaneously for Path D. Each truck stopped for approximately 10 s with its back midaxle, positioned at five locations or load cases, as follows: (1) near the abutment, (2) quarter span, (3) midspan, (4) three-quarter span, and (5) near the far abutment. The truck then went off the bridge, turned around, and repeated the process on the opposite side of the bridge for a total of 10 load cases for each load path (Fig. 4). Fig. 5 provides an example of Path D, Case 8 where the load case axle is positioned at midspan. All load paths were repeated 2x. Trial tests indicated that leaving the truck engine on yielded the same displacements and strains as turning it off for each load case. As such, the truck simply idled while stopped at each position.

The bridge was instrumented with strain gages and string potentiometers (string pots) primarily along the midspan axis and the west abutment. For the first repetition of the load paths, seven strain gauges were mounted on the midspan. A total of 38 strain gages were mounted on the girders. The deck was instrumented with string pots along the midspan axis and the west abutment. A total of 13 string pots were mounted on the girders. The deck was instrumented with strain gages along the midspan axis and the west abutment. A total of 38 strain gages were mounted on the girders. The deck was instrumented with string pots along the midspan axis and the west abutment. A total of 13 string pots were mounted on the girders.
gages measured the longitudinal and transverse strains on the FRP deck near the abutment and at midspan (Fig. 6). Maximum FRP strains on the order of 300 \( \mu \varepsilon \) (3% of ultimate capacity) were recorded transversely between girders 4 and 5 at midspan, and girders 7 and 8 at the abutment. During the second load path repetitions, nine strain gages measured longitudinal girder strains at midspan with a maximum value of approximately 250 \( \mu \varepsilon \) registered on the bottom of fascia girder 1. During all load path repetitions, five string pots measured vertical relative deck displacements and global girder displacements at midspan, with an additional deck displacement measurement at one of the abutments. A maximum girder deflection of 9.2 mm was recorded at midspan on girder 3. The maximum deck deflection measured was 1.75 mm on the abutment string pot.

**Validation of FE Model with Field Results**

The behavior of the rehabilitated bridge has been studied using the FE method to further observe the FRP deck and girder interactions. For this purpose, the FRP deck and girders are modeled using the commercial software Abaqus 6.12 (Simulia 2012). The FE model has been built by making two different parts, as follows: (1) FRP deck, and (2) steel girders. The FRP deck is modeled using “S4R” composite shell elements from the material library of Abaqus 6.12. The thickness, the number of integration points, and the orientation of each layer of the composite section for the horizontal walls, vertical walls, and outer wrap of the FRP deck have been applied in the FE model according to the material properties, which were obtained from coupon testing conducted in LeTourneau University (O’Connor 2013a; Fig. 1 and Table 1). The steel girders are modeled using C3D8R solid elements from the material library of Abaqus 6.12. The steel elastic modulus and Poisson’s ratio were assumed to be 200 GPa and 0.3, respectively. The details of the FE mesh are shown (Fig. 7). The model contained 885,172 elements and 1.044405 million nodes. All of the models were run using a Linux cluster with 72 processors.

The as-built support connections of the bridge show that the abutments support 356 mm of each end of the girders and that diaphragms using x-braces between girders are located at each end of the bridge. Furthermore, the girders are bolted to the bridge back wall, which prevent the girders from upward displacements and thus reduces the rotation of the end of the girders. Various types of boundary conditions were examined in the Abaqus model to mimic the effect of the as-built support connections and a pin/roller

<table>
<thead>
<tr>
<th>Table 1. Material Properties of the Composite Deck</th>
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<tbody>
<tr>
<td>Laminate unit value</td>
</tr>
<tr>
<td>---------------------</td>
</tr>
<tr>
<td>( E_x ) of 0(^\circ)</td>
</tr>
<tr>
<td>( E_x ) of 90(^\circ)</td>
</tr>
<tr>
<td>( G_{xy} )</td>
</tr>
<tr>
<td>Tensile strength of 0(^\circ)</td>
</tr>
<tr>
<td>Tensile strength of 90(^\circ)</td>
</tr>
<tr>
<td>Compressive strength of 0(^\circ)</td>
</tr>
<tr>
<td>Compressive strength of 90(^\circ)</td>
</tr>
<tr>
<td>Shear strength</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
</tr>
</tbody>
</table>

*Derived analytically.*
connection which restrains 178 mm of each end girders provides closely comparable results with the experimental data. In the pin connection, the nodal displacements of the boundary surface are restrained in all of the directions, while in the roller connection there is no restraint on the longitudinal displacement. A tie connection is used for modeling the interaction between the FRP deck and the girders. This interaction provides a continuous connection between the two surfaces. The diaphragms are not modeled in the FE model; however, the tie connection between the FRP deck and the girders prevents the out-of-plane deformation of the webs.

Experimental and analytical midspan steel girder strains (Fig. 8 shows gage locations) are given (Table 2). There is reasonably close agreement between the data for all strain gages. While some of the percent differences seem large (greater than 30%), the experimental strains at these instances are all small (less than 70 με), except SG9 at Load Path D, Case 8. For instance, analytical results indicate that strains measured during Path A, Case 8 (SG15) are almost half of the corresponding experimental strains. However, this value represents a 12-με difference between the two data points. Therefore, it is concluded the FE model accurately models the steel girder behavior.

Similarly, FRP deck experimental and analytical strains (Fig. 9 shows gage locations) are listed (Table 3). Similar to the steel girders, FRP analytical strains are reasonably close to the field data. Some of the large variances may be attributed to relatively small strain measurements (such as SG8 in Load Path A, Case 8) but more likely the differences can be attributed to the low stiffness of the deck. Even though special care was taken to align the trucks into the specified longitudinal and transverse locations on the bridge deck, small differences in wheel location could substantially change the measured bottom deck strain. This is especially true during Path D where both trucks were positioned with minimal space between them. As such, while the analytical FRP deck strains may vary from the field measured responses, they are clearly within the correct order of magnitude. Therefore, while strains at specific deck locations may not reflect the precise measured response, the FE model can be used to observe the FRP deck behavior in a general manner.

Analysis of Composite Action

The AASHTO (2012) LRFD bridge design specifications states that a deck should be composite with its girders where technically feasible, unless the deck is constructed with timber or open steel grating. Composite action is achieved by the deck to girder connection system which should eliminate relative movement between the deck and girders. Doing so can provide additional longitudinal load transfer capacity through the deck, thus increasing the effective girder stiffness. Different FRP deck to girder connections have been explored including shear studs embedded in grout, adhesive bonding, and bolted connections. The degree of composite action achieved can vary widely depending on the deck to girder connection which restrains 178 mm of each end girders.
connection system. It is most consistently obtained when shear studs are embedded in grout (Fu et al. 2007; Turner et al. 2004; Wan et al. 2005), although Reising et al. (2004) reports no composite action with this type of connection. Additionally, Moses et al. (2006) reports degradation of the apparent level of composite action over time in shear studs embedded in grout, suggesting that FRP decks should be conservatively designed to only carry loads transversely. Adhesive deck to girder connections studied by Keller et al. (2005) and Reising et al. (2004) have yielded composite and noncomposite decks respectively while bolted connections are mostly classified as noncomposite (Berman and Brown 2010; Zhang and Cai 2006). Park (2009) reports a decrease in composite action with increased bolt spacing in bolt-connected decks. Overall, it is apparent that the level of composite action achieved by the deck to girder connection varies widely and should be established for each unique connection type.

Fig. 10 shows the measured midspan relative deck deformations (between girders 4 and 5) and the bottom girder deflection of adjacent girder 5 during Load Path D. The girder deflection increases or decreases in steps as each of the two trucks approaches and moves beyond the bridge midspan showing the bridge global deflections at this location for each of the 10 load cases. The largest girder deflections (approximately 9 mm) were obtained with the trucks positioned at midspan (Cases 3 and 8). By contrast, the deck deflections measured relative to the girders show significant changes only when an axle is positioned directly over the string pot (shown by the small spikes on the relative deck deflection curve at Cases 3 and 8). This same pattern is observed when adjacent FRP and girder strain measurements are compared. The experimental data showed that FRP strains are insignificant unless the truck axle is positioned directly over the gages. Peak values around 315 με were recorded by abutment strain gage 2 during Load Case 6. At midspan, peak strains around 275 με were recorded during Load Case 8. Data from the FE model reinforces these observations. Fig. 11 provides the underside longitudinal FRP strains directly under one of the load case wheels during Load Path A, Case 8. Strains are greatest directly under the wheel loads. Similar to the field observations, FRP strain quickly transitions to a zero stress state as the distance from the applied load increases, even though the deck and the girders have a continuous connection in the FE model. Both field and analytical results indicate that the FRP deck distributes live loads only locally to the girders and not longitudinally through the deck due to a lack of composite action with the type of deck to girder connection assembly used.

Fig. 12 presents representative experimental strain profiles with the bridge under the loading case of two trucks positioned at midspan. The experimental neutral axes are all within 10% of the girder midheight indicating that little to no force is transferred longitudinally through the deck. To further determine the possibility of composite action, the FE model was built utilizing composite tie connections between the girders and deck. The FE model strain

<table>
<thead>
<tr>
<th>Truck position</th>
<th>Path A, Case 8</th>
<th>Path D, Case 3</th>
<th>Path D, Case 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>SG1</td>
<td>26</td>
<td>42</td>
<td>23</td>
</tr>
<tr>
<td>SG2</td>
<td>21</td>
<td>32</td>
<td>15</td>
</tr>
<tr>
<td>SG3</td>
<td>232</td>
<td>-62</td>
<td>-13</td>
</tr>
<tr>
<td>SG4</td>
<td>90</td>
<td>166</td>
<td>182</td>
</tr>
<tr>
<td>SG5</td>
<td>176</td>
<td>229</td>
<td>243</td>
</tr>
<tr>
<td>SG6</td>
<td>147</td>
<td>66</td>
<td>99</td>
</tr>
<tr>
<td>SG8</td>
<td>10</td>
<td>75</td>
<td>68</td>
</tr>
</tbody>
</table>

Table 3. Experimental and Analytical FRP Deck Strains
profiles, also plotted (Fig. 12), are within 10% of girder midheight. It can be concluded that the composite action of the bridge would be negligible even when it is assumed to have a continuous composite connection between the girders and the FRP deck. Visual inspection of the deck assembly showed gaps between the uneven surfaces of the painted girders, hardwood shims, and FRP deck, thus continuous composite action is very unlikely. It can be inferred that bridges using this new FRP deck and bolted connection are not likely to develop composite action with the girders. Owners should assess whether having a noncomposite deck is an acceptable design outcome when the FRP deck is used to replace an original composite deck.

Analysis of Distribution Factors

The AASHTO (2012) LRFD specifications allow for the deck on beam bridges to be designed based on live load distribution factors (DFs) which reduce the three-dimensional superstructure to line girders. The DF is defined as the fraction of the design loads on one design traffic lane carried by each girder, with the maximum fraction of load on a single girder taken as the design DF for all girders. In this way, the girders can be designed independent of the transverse live load location.

A low stiffness bridge deck will transfer a greater portion of the truck loads to the girder nearest to the wheels therefore requiring different DFs than a deck made of a high stiffness material. AASHTO (2012) provides DF equations for concrete, timber, and open steel grate decks supported on a variety of girder types. The timber and open steel grate DFs are calculated as a fraction of the span while the more complex concrete DFs are based on research by Zokaie (2000) who concluded that girder spacing has a significant influence on the DF. Therefore, many of the DF equations are limited to certain spans. When the spacing is outside the applicable range, AASHTO (2012) states that the lever rule can be used to calculate design DFs. The lever rule is a static simplification assuming the deck is hinged over all girders except at the exterior girder, such that a wheel load is distributed only to the two nearest girders. The design DF is found by applying point wheel loads at different transverse deck locations and summing the moments about the hinge such that the greatest girder vertical reaction becomes the design DF. Multiple presence factors must be taken into account when using the lever rule. Finally, for decks with diaphragms, AASHTO (2012) requires the exterior girder DF to also be determined with the rigid cross section formula [Eq. (1)], where \( N_b \) is the number of loaded lanes; \( N_g \) is the number of beams; \( X_{ext} \) is the distance from the exterior beam to the centroid of all girders; \( e \) is the eccentricity of the load to the girder centroid; and \( x \) is the distance of each girder to the centroid. The larger DF from the specification DF equations, including Eq. (1), is considered the DF to be used for design

\[
DF_{ext} = \frac{N_b}{N_g} + \frac{X_{ext} \sum N_g}{\sum X_g x^2}
\]  

There are no guidelines for the design of FRP decks in AASHTO (2012); however, it is possible to determine if the current DF provisions are applicable by comparing measured field responses to the design DFs for concrete, timber, or open steel grating. Field DFs are calculated using Eq. (2), where \( DF_i \) is the distribution factor at girder \( i \); \( \varepsilon_i \) is the bottom girder strain at \( i \); \( N \) is the number of loaded lanes; and \( \sum \varepsilon_j \) is the sum of all girder strains at the same location as \( \varepsilon_i \)

\[
DF_i = \frac{N \times \varepsilon_i}{\sum_{j=1}^{n} \varepsilon_j}
\]  

Different FRP decks have been studied using the approach described previously with varied results. A few studies have found that DFs for concrete deck were not conservative when compared to field tests of pultruded Duraspan and Strongwell decks on steel girders (Turner et al. 2004; Moses et al. 2006; Liu et al. 2008). Conversely, Fu et al. (2007) found the concrete LRFD equations to be appropriate for a glass fiber-reinforced polymer (GFRP) sandwich deck. Zhang and Cai (2006) showed that decreased composite action between the girders and the FRP deck decreased the girder stiffnesses, thus increasing the portion of live load to the girders nearest to the wheel loads. The higher loads required greater DFs for design. It is therefore important to establish the validity of the distribution factor equations for each unique FRP deck design.

Longitudinal midspan girder strain measurements (Fig. 8) show strain gage locations were used in conjunction with Eq. (2) to find the field DFs, assuming symmetric response about the bridge centerline: response measured with the truck in the opposite lane from the instrumented girders was mirrored to obtain the complete 11 girder DF profiles plotted (Fig. 13). The mirrored girder responses under Load Paths B and C [Figs. 13(b and c)] show that the girderarest from the truck position have consistently the lowest DF. This confirms the assumption that the superstructure behaves symmetrically about its longitudinal centerline, and that the results [Figs. 13(a and d)] are valid. Additionally, the nonsymmetric Cross-Load Paths B and C [Figs. 13(b and c)] show that the bridge diaphragms provided enough stiffness such that the rigid cross-section equation [Eq. (1)] suggested by AASHTO is valid.

The DF results (Fig. 13) show similar DFs when the truck or trucks have all three axles on the bridge (Cases 1–4). The truck front axles are on the bridge and the rear axles are located over the abutment (Cases 5 and 10), the data shows higher load distribution to the girders closest to the wheels. This is clearly observed in Fig. 13(d) where the Case 5 DFs are significantly greater for girders 4, 5, 7, and 8 than those seen during cases 1–4. With limited load over the clear span of the bridge, load transfer to adjacent girders relies more on the flexible FRP deck and less on the midspan diaphragms. With less contribution from the diaphragms, there is less transverse load distribution than when the trucks are at midspan, and hence a higher portion of loads is distributed to those girders closest to the wheels.

Noting that the Pleasant Street Bridge has a 0.61-m girder spacing, the AASHTO (2012) design DFs for concrete, timber, and open steel grate decks were evaluated. The bridge girder spacing does not meet the criteria for the concrete DF equations; 1,100 mm < S < 4,900 mm, hence the lever rule must be used in order to establish an appropriate DF in that case. For timber and open steel grate decks, simple fractions of girder spacing DF equations are given. The various AASHTO DF equations used are summarized (Table 4). The lever rule without multiple presence factors yields a much more conservative result (0.50 lanes/girder) than the timber and open steel grate DFs (0.23 and 0.20 lanes/girder, respectively). For exterior girders, both the lever rule and rigid cross-section formula DFs were calculated with the later governing (0.21 lanes/girder).

The field DFs were compared to the AASHTO calculated limits (Table 4). The lever rule is always conservative when the DFs are calculated at the tested load configurations for interior girders for the bridge under study. Additionally, the timber and open steel grate DFs are nonconservative when compared to Load Path C and D maxima. Fig. 14 displays the field results yielding the highest calculated DFs, 0.34 trucks/girder on girders 5 and 7, measured...
during Load Path D, Case 5. The values are compared to the AASHTO limits for the lever rule, timber decks, and open steel grate decks. The simple s over factors for timber and open steel grate decks are not consistently conservative to be used for design purposes but the lever rule proves to be adequate for design when the AASHTO prescribed geometry is used. Therefore, the lever rule is recommended to estimate the DFs for this new FRP deck at 0.61-m girder spacing. The rigid cross section formula provides a conservative estimate of load distribution to exterior girders (0.21 trucks/girder) for all load paths except Path B. AASHTO (2012) specifies the truck to be positioned no closer than 0.61 m from the inside of the bridge rail when applying the rigid cross-section formula while Load Path B positions the outer wheel within a few centimeters of the rail. Therefore, as the difference between the field results and AASHTO (2012) specifications is small, the rigid cross section is recommended where applicable when using the tested bridge configuration.

The original design of the deck was intended to have a grouted cross section and be used on bridges with girder spacing up to 1.6 m. The FE analysis showed that for the current bridge girder spacing (0.61 m) a nongrouted deck cross section did satisfy weight

<table>
<thead>
<tr>
<th>Girders</th>
<th>DF</th>
<th>Load path (trucks/girder)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td>Interior</td>
<td></td>
<td>0.14</td>
</tr>
<tr>
<td>Lever rule at tested position&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.25</td>
<td>0.30</td>
</tr>
<tr>
<td>AASHTO geometry lever rule&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Timberdeck</td>
<td>0.23</td>
<td></td>
</tr>
<tr>
<td>Open steel grate deck</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>Exterior</td>
<td></td>
<td>0.17</td>
</tr>
<tr>
<td>Max field</td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>Rigid cross section</td>
<td>0.21</td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup>No multiple presence factors included.

**Fig. 13. Field measured DFs**

**Table 4. Field versus AASHTO Design Moment DFs**

![Fig. 14. AASHTO limits versus maximum field results (Path D, Case 5)](image-url)
and performance criteria. To further explore the performance of the nongrouted deck, AASHTO (2012) DFs were evaluated up to a value of 1,676 mm for girder spacing. For girder spacing in the range of 1,100–4,900 mm, AASHTO (2012) specifies more detailed formulas, developed by Zokaie (2000), to relate a variety of variables to the load distribution. Eqs. (3) and (4) give DF values for one and two lanes loaded, respectively, where \( S \) is the girder spacing (mm); \( L \) is the bridge clear span; \( K_2 \) is a longitudinal stiffness parameter; and \( t_r \) is the depth of the concrete slab (mm).

One lane loaded, \[ DF = 0.06 + \left( \frac{S}{4300} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_2}{L_t} \right)^{0.1} \] (3)

Two lane loaded, \[ DF = 0.075 + \left( \frac{S}{2900} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_2}{L_t^2} \right)^{0.1} \] (4)

The DFs for two hypothetical bridges with 1,067 and 1,676-mm girder spacing, respectively, were calculated using Eqs. (3) and (4), assuming the same slab thickness as the deck, 114 mm, and 27.6 MPa (typical) concrete compressive strength, similar to the approach used in Liu et al. (2008). Additionally, open steel grate, timber, and lever rule DFs were calculated for a 1,067 and 1,676-mm girder spacing with the methods previously outlined. Theoretical results from these calculations (and the maximum experimental value obtained from the field test at 610-mm spacing) are shown (Table 5). At 1,067-mm spacing, a small increase on the load distribution of 0.15 trucks/girder over 610-mm spacing measurements would render all but the lever rule nonconservative for predicting the maximum load resisted by a girder. At 1,676-mm spacing, the largest load distribution to a girder would likely exceed the value predicted by the AASHTO (2012) timber, open steel gate, and concrete equations.

Therefore, based on the analysis carried out in this paper, it is recommended that the lever rule be used for all interior girders spaced up to 1,676 mm when using this new FRP deck. Additionally, the rigid cross-section formula [Eq. (1)] provides a conservative estimate of the exterior girder DFs for the bridge geometry studied.

**Analysis of Girder Serviceability**

In 1936, the U.S. Bureau of Public Roads established the common \( L/800 \) deflection limit for steel highway bridges carrying light traffic in order to reduce both the structural effects due to dynamic stresses and the psychological effects of bridge vibrations. ASCE (1958) found the limit to be sufficient only to reduce unwanted vibrations as no major structural damage or safety concerns can be directly related to deflections exceeding this limit. The same deflections limits have remained unchanged since they were codified. Because the design of FRP decks is primarily controlled by deflection due to its high elastic modulus to weight ratio (Demitz et al. 2003), designers often revert to an \( L/800 \) limit. Researchers have indicated that if less restrictive deflection limits are imposed on FRP deck designs, the material strength capacity can be used more efficiently therefore increasing the economic competitiveness of the deck (Demitz et al. 2003; Zhang and Cai 2006).

For the Pleasant Street Bridge with a 12.2-m span, the AASHTO (2012) \( L/800 \) limit for girder deflection of 15.24 mm is greater than the largest field measured girder deflection of 9.20 mm (not including dynamic allowance). AASHTO (2012) states that this limit is imposed to prevent damage to the wearing surface and to increase user comfort. The Ipanol E-Flex wearing surface used in this project has a 50% elongation tensile elongation capacity (IPA Systems 2011) which should limit damage due to excessive deformations. On the other hand, user comfort is a function of the bridge’s dynamic properties (Wright and Walker 1971) and the widely varying human response to accelerations (Wright and Green 1959). Several researchers have indicated that a simple deflection check does not adequately limit vibrations and therefore serviceability requirements should be related to allowable accelerations and the dynamic properties of the bridge (Wright and Walker 1971; Roeder et al. 2002; Demitz et al. 2003; Barker and Barth 2013). Therefore, the bridge under study was compared against the serviceability requirements given in Wright and Walker (1971), also cited in AASHTO (2012), and the serviceability requirements found in both CAN/CSA-S6-00 (Canadian Standards Association 2000) and Ontario Bridge Design Code (Ministry of Transportation 1991).

Wright and Walker (1971) developed a procedure to calculate the bridge natural frequency and proposed an acceleration limit equal to 2.54 m/s\(^2\) to prevent accelerations in the same range as those expected when using an \( L/800 \) limit. The natural frequency for a simply supported girder is calculated using Eq. (5), where \( f_b \) is the natural frequency; \( L \) is the span length; \( E_d I_b \) is the beam flexural rigidity with a composite deck; \( g \) is gravitational acceleration; and \( \omega \) is the beam section unit weight (girder plus deck).

\[ f_b = \frac{\pi}{2L^2} \sqrt{\frac{E_d I_b g}{\omega}} \] (5)

The bridge girder noncomposite natural frequency \( f_b \) was 9.25 Hz via Eq. (5), using standard W530X101 bridge girder dimensions, and assuming that the steel elastic modulus is 200 GPa. With the Wright and Walker (1971) procedure, the maximum field measured static girder deflection of 9.20 mm and assuming a truck is traveling at the posted 40 km/h speed limit, the calculated girder acceleration is 6.25 m/s\(^2\). This is about 2.5\( \times \) greater than the proposed acceleration limit. While Wright and Walker (1971) state that the proposed acceleration limit should be verified with empirical data, this analysis indicates that girder accelerations for the bridge under study may be greater than comfortable human limits.

Rather than a constant acceleration limit, both CAN/CSA-S6-00 (Canadian Standards Association 2000) and Ontario Bridge Design Code Ministry of Transportation (1991) codes convert allowable accelerations to equivalent static deflections, then limit the deflection based on the bridge natural frequency. A series of curves set the deflection limits such that bridges with lower natural frequencies and less pedestrian use have a greater allowable static deflection (Fig. 15). Using the same natural frequency (9.25 Hz) calculated with Eq. (3), and the maximum recorded girder deflection (9.20 mm), the acceleration of the bridge exceeds the limits set.

**Table 5. AASHTO Design Interior Moment DFs at Greater Girder Spacings**

<table>
<thead>
<tr>
<th>Girder spacing (mm)</th>
<th>Calculation method</th>
<th>Value (trucks/girder)</th>
</tr>
</thead>
<tbody>
<tr>
<td>610</td>
<td>Max field test</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>Timber</td>
<td>0.39</td>
</tr>
<tr>
<td>1,067</td>
<td>Open steel grate</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>Lever rule</td>
<td>0.50</td>
</tr>
<tr>
<td>1,676</td>
<td>Concrete</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Timber</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>Open steel grate</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Lever rule</td>
<td>0.64</td>
</tr>
</tbody>
</table>

Comparison of girder field response versus AASHTO limit and Canadian serviceability standards

by CAN/CSA-S6-00 (Canadian Standards Association 2000) and Ontario Bridge Design Code Ministry of Transportation (1991) codes even though it is less than the constant AASHTO deflection limit.

Both Wright and Walker (1971) and CAN/CSA-S6-00 (Canadian Standards Association 2000) suggest that the girder accelerations for the bridge under study exceed comfortable human limits. It is expected that the relative light weight of the FRP deck when compared to the original 50-mm open steel grate deck allows for greater vibration. Fiber-reinforced polymer decks are used mainly for bridge rehabilitations to reduce the bridge dead load thereby increasing the allowable live load. In these circumstances, changing the girder size to account for the change in bridge stiffness is cost prohibitive. Therefore, bridge owners should expect greater bridge girder deflections and accelerations when replacing a conventional deck with this lightweight FRP deck. In conclusion, the AASHTO L/800 deflection limit may not be appropriate to govern bridge serviceability related to girder accelerations. Instead, accelerations should be limited by relating the girder natural frequency and allowable limits of acceleration or deflection, such as the limits proposed by Wright and Walker (1971) or in the CAN/CSA-S6-00 (Canadian Standards Association 2000) and Ontario Bridge Design Code Ministry of Transportation (1991) codes.

Analysis of Deck Serviceability

Since FRP deck design is primarily governed by deflection concerns, the only deck deflection criteria within AASHTO (2012) is the metal grid deck deflections between girders to S/800. This specification is, once more, designed to protect the wearing surface and to limit excessive vibrations. The maximum measured relative deck deflection between girders was 1.75 mm directly under a 1,676-mm girder spacing for the tested FRP deck. This value exceeds the AASHTO S/800 limit of 0.76 mm at 0.61-m girder spacing. The timer and open steel grate deck DF equations under estimate the maximum field response at 0.61-m girder spacing. The timber and open steel grate deck DF equations under estimate the maximum field response at 0.61-m girder spacing. The timber and open steel grate deck DF equations under estimate the maximum field response at 0.61-m girder spacing. The timber and open steel grate deck DF equations under estimate the maximum field response at 0.61-m girder spacing.

Comparisons between distribution factors based on field measurements and AASHTO recommendations show that the lever rule yields interior moment design distribution factors 1.8× greater than the maximum field response at 0.61-m girder spacing. The timber and open steel grate deck DF equations underestimate the maximum field response by 63% and 60%, respectively. Results indicate that the lever rule is consistently conservative to estimate load distribution up to a 1,676-mm girder spacing for the tested FRP deck.

Conclusions

This paper presents results from a field test and finite-element modeling of a new FRP deck supported by steel girders. Data gathered was analyzed and compared against AASHTO (2012) LRFD specifications. Major conclusions are as described next.

The FE modeling method satisfactorily predicts the steel girder behavior when supporting an FRP deck. The strain values and the behavior of the FRP deck can also be predicted using the FE model. However, some of the strain quantities under the truck loading cannot be captured well.

Experimental and analytical data indicate that the new FRP deck used on the Pleasant Street Bridge deflects locally under wheel loads while global deformations are reflected in girder deflections, thus indicating that the wheel load is primarily transferred transversely through the deck. Field observations and strain profiles, along with FE simulation, indicate that no composite action is developed by the deck-girder connection. Bridge owners should therefore assess whether a noncomposite deck could provide the necessary bridge performance.

Comparisons between distribution factors based on field measurements and AASHTO recommendations show that the lever rule yields interior moment design distribution factors 1.8× greater than the maximum field response at 0.61-m girder spacing. The timber and open steel grate deck DF equations underestimate the maximum field response by 63% and 60%, respectively. Results indicate that the lever rule is consistently conservative to estimate load distribution up to a 1,676-mm girder spacing for the tested FRP deck.

Even though measured girder deflections are less than the AASHTO L/800 limit, girder accelerations were calculated to be 2.5× greater than the acceleration limit suggested by Wright and Walker (1971). The 9.20-mm girder deflection is approximately 1.3× greater than the limit imposed by CAN/CSA-S6-00 (Canadian Standards Association 2000) and the Ontario Bridge Design Code Ministry of Transportation (1991). It is recommended that by relating the dynamic properties of the bridge and allowable accelerations, limits can be established to appropriately govern girder serviceability. Owners should expect greater deflections.
and accelerations when replacing a conventional concrete, timber, or open steel grate deck with the new FRP deck. The AASHTO S/800 limit for relative deck deflections does not properly address pedestrian comfort. Numerical data indicates that the FRP deck deflections, and by extension accelerations, are insignificant in areas where pedestrians are likely to stand near passing traffic. Therefore, the new FRP deck effectively limits accelerations felt by pedestrians, and its serviceability performance in this regard should be considered acceptable.

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References


Simulia. (2012). ABAQUS 6.12 documentation, Dassault Systèmes Simulia Corporation, Providence, RI.


Wright, D. T., and Green, R. (1959). Human sensitivity to vibrations, Queen’s Univ., Kingston, ON, Canada.
