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REPLACING A COMPOSITE RC BRIDGE DECK WITH AN FRP DECK – THE EFFECT ON SUPERSTRUCTURE STRESSES

K.A. Harries[1*] and J. Moses[2]

[1] *Department of Civil and Environmental Engineering,*
University of Pittsburgh, USA. Email kharries@engr.pitt.edu
[2] *Pennsylvania Department of Transportation, Bridgeville, PA USA*

ABSTRACT

Glass fibre reinforced polymer (GFRP) composite bridge decks behave differently than comparable reinforced concrete (RC) decks. GFRP decks exhibit reduced composite behaviour (when designed to behave in a composite manner) and exhibit reduced transverse distribution of forces as compared to comparable RC decks. Both of these effects are shown to counteract the beneficial effects of a lighter deck structure and result in increased internal stresses in the supporting girders. The objective of this paper is to demonstrate through an illustrative example the implications of RC-to-GFRP deck replacement on superstructure stresses. Regardless of superstructure stresses, substructure forces will be uniformly reduced due to the resulting lighter superstructure.

KEYWORDS

composite section, GFRP bridge deck, rehabilitation, steel girder.

INTRODUCTION

Glass fibre reinforced polymer (GFRP) composite bridge deck systems are light-weight and durable alternatives to reinforced concrete (RC) decks as well as non-porous alternatives to unfilled steel grid-deck. Although most existing demonstration projects in North America are based on new bridges, GFRP decks hold their greatest promise as a means of deck replacement for older, deteriorated or specialised structures. Indeed, such decks have found use as replacement decks on historic and deteriorated truss structures and as light-weight yet solid (non-porous) alternatives to steel grid-deck on bascule bridges. GFRP bridge decks capable of replacing 200 to 250 mm thick RC decks, weighing 4.8 – 6 kN/m^2, typically weigh about 1 kN/m^2 without a wearing surface; thus representing a significant reduction in the dead load of the superstructure.

Extensive *in situ* testing of three GFRP deck-on-steel girder bridges (BDI 2003, Luo 2003 and Turner 2003) reported by Moses et al. (2005) identified the following differences between the assumed behaviours of RC decks and GFRP decks:

The effective width of GFRP deck which may be engaged in a composite manner with a supporting steel girder is less than that of an equivalent RC deck. This decrease results from increased horizontal shear lag (across the width of the deck) due to the less stiff axial behaviour of the GFRP deck as compared to RC and increased vertical shear lag (through the depth of the deck). The latter results in a reduced contribution of the top face plate, due to the relatively soft in-plane shear stiffness of the GFRP deck (Keller and Gurtler 2005). Additionally, the engaged effective width shows evidence of degradation with time; a condition that is most likely related to a reduction in the shear transfer efficiency required to develop composite behaviour. The composite behaviour exhibited indicated that while a degree of composite action is achievable at service load levels, it is likely that composite action will be exhausted at ultimate load levels. Thus it was proposed (Moses et al. 2005) that composite behaviour should be conservatively neglected when designing GFRP deck-on-steel girder bridges. Nevertheless, a reduced degree of composite action may be engaged and thus is appropriate when service load design (deflection) checks are made. Moses et al. recommend that for this latter case, composite section properties may be calculated using an effective width of GFRP deck equal to 75% of that calculated for an RC deck having the same depth; this factor accounts for both horizontal and vertical shear lag effects.

GFRP decks also exhibited reduced transverse distribution of wheel loads from girder to girder across the deck width resulting in increased moment distribution factors (used to assess design forces in girders). This effect results from the increased transverse flexibility of the GFRP deck system as compared to an equivalent RC deck. There is insufficient data to recommend a distribution factor calculation for GFRP deck systems consistent with those prescribed by AASHTO (2004) for RC decks. Therefore, it was proposed (Moses et al. 2005) that appropriately conservative design values may be found using the "lever rule" to determine transverse load distribution. That is, loads are distributed statically only to adjacent girders.

The implications of these findings are that GFRP decks behave in a fundamentally different manner than RC decks. The reduced dead load resulting from an RC-to-GFRP deck replacement may not necessarily translate to reduced forces and stresses throughout the bridge structure. The objective of this paper is to demonstrate the implications of RC-to-GFRP deck replacement on superstructure stresses. It is also demonstrated that, generally, substructure forces will be uniformly reduced due to the lighter resulting superstructure.

BRIDGE DECK REPLACEMENT WITH GFRP DECK – ILLUSTRATIVE EXAMPLE

In a bridge deck replacement scenario, a reinforced concrete (RC) deck may be replaced with a GFRP deck. The original concrete deck will typically be composite with the supporting steel girders. The replacement GFRP deck may be composite or non-composite with the girders. Certainly by replacing a concrete deck with a GFRP deck, the structural dead load is reduced. Nonetheless, as a result of the decreased composite action and the reduced transverse distribution of forces described above, the resulting stresses in the supporting girders may, nevertheless, increase. As will be shown in the following sections, this is particularly the case for the compression flange of the supporting girder as there may be a significant downward shift in the neutral axis of the composite girder section when replacing an RC deck with GFRP.

The magnitude of live load is unaffected by deck replacement. However, live load-induced transient stresses in the supporting girders are increased due to the reduced composite action and lateral distribution of loads. This increased transient stress range must be of concern if fatigue-prone details are present. The following sections present an illustrative example, based on the geometry of an existing GFRP-decked bridge (Turner 2003 and Turner et al. 2004), which assesses girder stresses prior to and following the replacement of an existing composite RC deck.

Prototype Bridge Geometry

The prototype bridge selected for this example is a 17.5 m long single span consisting of five W760x185 girders having a transverse spacing of 2440 mm as shown in Figure 1. The existing bridge has a 216 mm deep RC deck acting compositely with the girders. This design is adequate to carry an AASHTO (2004) HS25 live load.

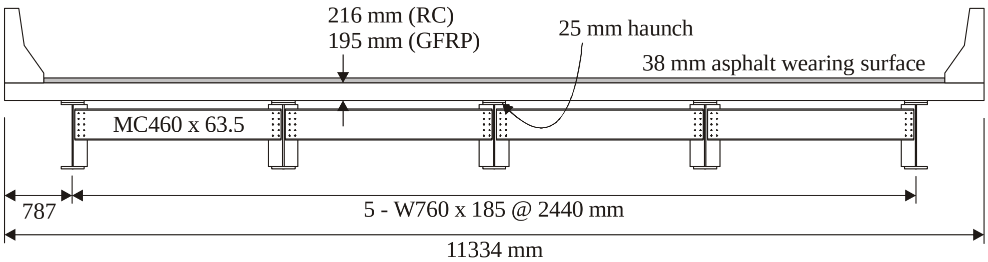


Figure 1. Prototype bridge section.

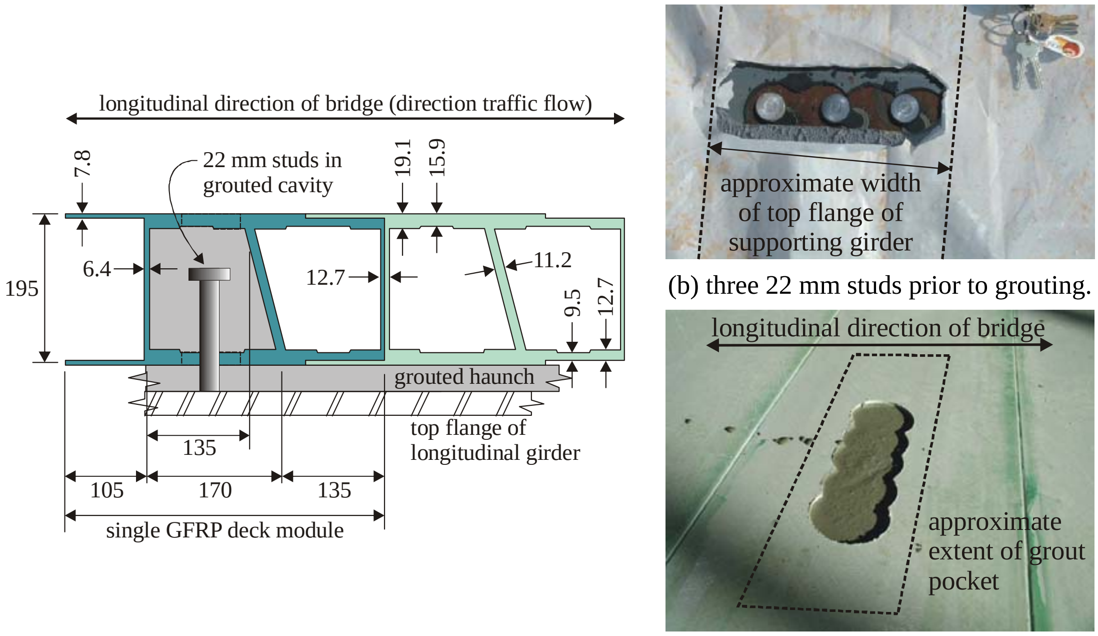
Deck Replacement Scenarios

The deck replacement scenario considered is one in which the existing composite RC deck is replaced with a GFRP deck system. It is often assumed, incorrectly, that composite RC bridge decks replaced with GFRP bridge decks remain fully composite. While fully composite behaviour may be exhibited for AASHTO (2004) SERVICE I and FATIGUE load cases, as discussed previously, it is unlikely that composite behaviour will be achieved under STRENGTH I conditions. Therefore it is appropriate and more conservative to assume non-composite behaviour of GFRP decks. Thus two deck replacement scenarios arise: a) Composite RC deck replaced with composite GFRP deck; and b) Composite RC deck replaced with non-composite GFRP deck.

GFRP Replacement Deck

The GFRP replacement deck considered in this study was selected because it is the most prevalent GFRP deck system in North America (Hooks and O'Connor 2004). The commercially available (MMC 2001) 195 mm deep

GFRP bridge deck panels are formed by assembling 305 mm long interlocking pultruded elements. The interlocking elements may be joined together to form any length of deck necessary. Figure 2(a) shows a 610 mm long section of this deck system made up of two interlocking elements. Full bridge-width preassembled panels (typically ten elements or 3.05 m in length) are delivered to the bridge site. Final placement, interlocking and bonding of the panels occurs on site. To develop composite action and to anchor the panels, three 22 mm diameter, 150 mm tall shear studs are located in grouted pockets spaced at 610 mm along the entire length of each girder as shown in Figures 2(b) and (c). It has been shown that such a shear connection is adequate to develop the composite behaviour discussed previously at SERVICE load levels (Turner et al. 2004, Keelor et al. 2002; Moon et al. 2002). Less shear connection (fewer studs per pocket) results in proportionally reduced composite behaviour (Moses et al. 2005).



(b) three 22 mm studs prior to grouting.

(a) Transverse section of GFRP deck. (c) stud pocket following grouting.

Figure 2. Prototype GFRP bridge deck (based on Turner 2003).

Prototype Bridge Sectional Properties

Table 1 provides assumed material properties and resulting member properties for a single interior girder for the prototype bridge. Three scenarios are required to assess the deck replacement scenarios:
Scenario 1: the existing 216 mm thick RC deck acting compositely with the supporting W760 x 185 girders.
Scenario 2: a 195 mm thick GFRP deck acting compositely with the supporting girders.
Scenario 3: the same GFRP deck acting non-compositely with the supporting girders.
All cases are assumed to include a 25 mm thick grout haunch between the deck soffit and girder (see Figure 2(a)).

For Scenarios 1 and 2, composite section properties were derived using transformed section properties and simple mechanics. For Scenario 1, the neutral axis of the composite section was calculated using an effective width for the RC deck of 2440 mm, equal to the girder spacing. For Scenario 2 an effective width of 75% of the Scenario 1 value, or 1830 mm was used (based on recommendation of Moses et al. 2005). The distribution factor used for subsequent load calculations was determined based on AASHTO 2004 recommendations for the RC deck (Scenario 1) and using the lever rule for the GFRP decks (Scenarios 2 and 3). Both the grout haunch and RC deck were assumed to have a compressive strength, f_c', of 27.6 MPa and a modulus of 24.8 GPa. The reported compressive strength of the GFRP deck is 172 MPa and the compressive failure strain is 0.013 (Turner et al. 2004), resulting in a compressive modulus, E_{FRP} equal to 13.2 GPa.

Applied Loading and Load Cases

All loads were determined based on AASHTO LRFD (2004) provisions. In this example, only the following vertical loads were considered:
DC – dead load of structural components, including deck, girders, diaphragms and parapets. The parapets assumed for the example were typical concrete "jersey barrier" parapets weighing 5.9 kN/m. Two MC460 x 63.5 diaphragms were assumed to be located between each girder at the bridge third points (see Figure 1).

DW – dead load of wearing surface. A 38 mm thick asphalt wearing surface weighing 0.84 kN/m^2 was assumed. Such a wearing surface has been provided on a number of GFRP decks including the model for the present example (Turner et al. 2004)
LL – vehicular live load. Impact loading (IM) equal to 0.33LL (0.15LL for FATIGUE load case) was also considered.

The moments applied to a typical interior girder resulting from these loads are given in Table 1. Three AASHTO 2004 LRFD load combinations were considered:

STRENGTH I:	1.25DC + 1.50DW + 1.75(1.33LL)	(1)
SERVICE I:	DC + DW + 1.33LL	(2)
FATIGUE:	0.75(1.15LL)	(3)

Table 1. Section parameters and applied loading.

	Scenario 1	**Scenario 2**	**Scenario 3**
deck type	composite RC	composite GFRP	non-composite GFRP
deck thickness and type	216 mm RC	195 mm GFRP	195 mm GFRP
deck weight	5 kN/m^2	0.96 kN/m^2	0.96 kN/m^2
effective width, b_{eff}	2440 mm	1830 mm	1830 mm
moment of inertia, I_x	6.894×10^9 mm^4	3.236×10^9 mm^4	2.231×10^9 mm^4
location of elastic neutral axis[1], $\bar{y}$	765 mm	467 mm	384 mm
interior girder distribution factor, DF	0.688 (AASHTO 2004)	0.875 (lever rule)	0.875 (lever rule)
		Girder Moments	
dead load of components, **DC**	659 kNm	271 kNm	271 kNm
dead load of wearing surface, **DW**	84 kNm	84 kNm	84 kNm
vehicle live load, **LL**	608 kNm	772 kNm	772 kNm

[1] measured from bottom of girder

Superstructure Stresses

Table 2 gives the maximum tension and compression stresses in steel girders for each of the load combinations considered. The stresses are calculated by simple mechanics using elastic section properties given in Table 1. In all scenarios, moments corresponding to DC loads are applied to the non-composite steel girder section. DW and LL loads are applied to the composite (Scenarios 1 and 2) or non-composite (Scenario 3) section as appropriate.

Stresses determined for Scenario 1 assume original conditions. *In situ* stresses at the time of deck replacement are likely greater assuming that the RC deck is no longer behaving in a fully composite manner or has reduced effectiveness due to deterioration of the concrete. It is also evident from Table 2 that adopting deck replacement Scenario 3 results in girder stresses greater than that permitted, $\phi_f F_y = 1.0 \times 345 = 345$ MPa, and would thus require additional retrofit of the superstructure.

Table 2. Superstructure stresses.

Scenario	**1**	**2**	**3**
		STRENGTH I	
compression flange	-142 MPa	-236 MPa	-389 MPa[1]
tension flange	313 MPa	336 MPa	389 MPa[1]
		SERVICE I	
compression flange	-114 MPa	-150 MPa	-238 MPa
tension flange	212 MPa	208 MPa	238 MPa
		FATIGUE (stress range)	
compression flange	-0.7 MPa	-61 MPa	-114 MPa
tension flange	59 MPa	97 MPa	114 MPa

[1] stress exceeds that allowed

Table 3. Ratios of loads and superstructure stresses after deck replacement to those before.

From Scenario 1 to…	**2**	**3**
Applied Loads and Load Combinations		
DC	0.41	0.41
DW	1.00	1.00
LL	1.27	1.27
STRENGTH I (Eq. 1)	0.96	0.96
SERVICE I (Eq. 2)	0.89	0.89
FATIGUE (Eq. 3)	1.27	1.27
Superstructure Stresses		
STRENGTH I		
compression flange	1.66	2.74
tension flange	1.07	1.24
SERVICE I		
compression flange	1.32	2.09
tension flange	0.98	1.12
FATIGUE		
tension flange	1.65	1.96

Effect of Deck Replacement Scheme

Table 3 shows the ratios of applied loads and girder stresses following deck replacement to those prior to deck replacement for Scenarios 2 and 3. As expected, the self weight of the bridge (DC) decreases substantially by replacing an RC deck with a GFRP deck. The transient live loads, however, increase due to the increased distribution factor (Table 1) used for the GFRP deck. The combination of these effects is that for STRENGTH I, an RC-to-GFRP deck replacement only reduces the applied girder moment 4%. At SERVICE I, this reduction is 11%. Since FATIGUE only considers transient loads, the applied load for this combination increases 27%.

The changes in applied load and section properties resulting from RC-to-GFRP deck replacement generally result in an increase in superstructure internal stresses. For replacement Scenario 2 at STRENGTH I, this increase is 66% for the compression flange and 7% for the tension flange of the steel girder. As previously stated, Scenario 2 results in an even greater increase resulting in the flange stress exceeding that allowed. These increases are not as significant at SERVICE I. FATIGUE stresses will be discussed in the following section.

It is often incorrectly assumed that composite RC bridge decks replaced by GFRP bridge decks remain fully composite (Scenario 2). This fully composite behaviour may be appropriate for SERVICE I and FATIGUE load cases. Nonetheless, as discussed previously, it is unlikely that composite behaviour may be achieved at STRENGTH I. Therefore it is appropriate and more conservative to assume non-composite behaviour (Scenario 3). As a result, there is an error associated with stresses determined assuming Scenario 2 when Scenario 3 is actually appropriate. In this example, compression flange stresses are underestimated by 39% and tension flange stresses by 14% at STRENGTH I and 37% and 13% at SERVICE I load combinations. FATIGUE stresses are underestimated by 16%.

AASHTO Fatigue Criteria

Fatigue loading is a special case requiring particular attention if fatigue-prone details are present. In AASHTO (2004), steel details are assigned a fatigue category ranging from Category A (best, having little susceptibility to fatigue damage) to Category E' (worst). Each category has a threshold stress range which is compared to the FATIGUE load case to assess the adequacy of the detail. For the example structure, the FATIGUE stress ranges are given in Table 2. Table 4 provides the AASHTO (2004) fatigue thresholds for a typical rural bridge having ADT = 1000 with 10% trucks. Also shown in Table 4 are the acceptable details for each of the three scenarios considered based on their calculated stress range (Table 2).

Table 4. AASHTO (2004) fatigue thresholds.

Scenario	**FATIGUE**	**A**	**B**	**B'**	**C**	**C'**	**D**	**E**	**E'**
	stress range	144 MPa	113 MPa	90 MPa	81 MPa	81 MPa	64 MPa	51 MPa	36 MPa
1	59 MPa	OK	OK	OK	OK	OK	OK	NG	NG
2	97 MPa	OK	OK	NG	NG	NG	NG	NG	NG
3	114 MPa	OK	NG	NG	NG	NG	NG	NG	NG

As seen in Table 4, the original structure having a composite RC deck, Scenario 1, provides adequate resistance for fatigue category D and better. When replacing the RC deck with a GFRP deck, the increased tensile flange stress results in poorer fatigue performance. Replacement Scenario 2 only provides adequate resistance for fatigue categories A and B while the Scenario 3 is adequate only for fatigue category A. Regardless of the replacement scenario, however, a degree of composite action (intentional or not) can likely be developed at the FATIGUE load level, thus the results for Scenario 2 may be considered representative of RC-to-GFRP deck replacement.

While fatigue resistance may not be a significant concern in the high-tensile stress regions of bridges fabricated from rolled shapes, other conditions may arise where this is not the case. For example, built-up sections having partial penetration groove welds connecting web to flange are fatigue category B' and welded details at the end of partial length cover plates are fatigue category E'. In the latter case, retrofit of the offending detail would likely be required in order to accommodate the increase stress accompanying an RC-to-GFRP deck replacement.

Rehabilitation of older or historic bridges may benefit from the use of lighter GFRP decks, however the older bridge details may also have fatigue-prone details. The original bridge which was replaced by that on which this example is modeled (Turner 2003), for instance, had built-up riveted girders; the entire length of such a girder would be classified as fatigue category D.

CONCLUSIONS

As can be seen in this single illustrative example the internal stresses in a bridge superstructure may be significantly affected when replacing a concrete deck with a GFRP deck. The more flexible axial behaviour of the GFRP deck results in reduced composite behaviour developed between the deck and superstructure resulting in a correspondingly lower neutral axis location. This effect is compounded by the reduced effective width of the GFRP deck. Similarly, the flexible deck does not permit as much transverse distribution of applied loads as an RC deck resulting in higher loads carried by girders immediate adjacent travel lanes. This effect is reflected in higher distribution factors for girder design. Finally, particular care must be taken when replacing the deck on structures having fatigue-prone details. Even though the dead load is reduced in an RC-to-GFRP deck replacement, the live load remains the same. Due to decreased composite action and transverse distribution, the resulting live load stresses in the supporting girders may increase. This increased transient stress range must be considered if fatigue-prone details are present.

When a GFRP deck replaces a concrete deck, it has been shown that superstructure internal stresses may increase – significantly in some cases. Nonetheless, forces on structural elements further along the load path (bearings, support bents, columns, abutments and foundations) are not as severely affected. As shown in relation to STRENGTH I and SERVICE I combinations in Table 3, the design loads will generally be reduced due to the significant reduction in deck weight. Additionally, many bridge components are unaffected by transverse force distribution. In such cases the reduction in design forces will be more pronounced (an additional 21% in the example cited).

Replacement of an RC bridge deck with a GFRP deck can result in significantly reduced loads, particularly on the bridge substructure. Great care must be taken, however as, despite the reduction in applied load, superstructure internal stresses may increase to a degree significant enough to exacerbate fatigue-prone details. Finally, it is cautioned that this example has focused on the use of a particular FRP deck system supported on steel girders. This system was chosen as it is currently represented on 25 of the 83 GFRP decked bridges in the United States. Properties of FRP deck systems vary greatly from system to system; no inference should be made from the present results beyond their admittedly limited scope.

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