UC Irvine

UC Irvine Previously Published Works

Title

Experimental and numerical investigation on assessing local bearing behavior of a pultruded GFRP bridge deck

Permalink

<https://escholarship.org/uc/item/4qt6g551>

Authors

Xin, Haohui
Mosallam, Ayman S
Liu, Yuqing
et al.

Publication Date

2018-11-01

DOI

10.1016/j.compstruct.2018.07.082

Peer reviewed

Experimental and numerical investigation on assessing local bearing behavior of a pultruded GFRP bridge deck

Haohui Xin^{a,b}, Ayman S. Mosallam^c, Yuqing Liu^{a,*}, Congzhe Wang^a, Jun He^d

^a Department of Bridge Engineering, Tongji University, Shanghai, China

^b Faculty of Geoscience and Engineering, Delft University of Technology, Netherlands

^c Department of Civil and Environment Engineering, University of California, Irvine, CA, USA

^d School of Civil Engineering, Changsha University of Science & Technology, Hunan, China

ARTICLE INFO

Keywords:

Pultruded GFRP deck
Local properties
Finite element simulation
Tire-to-deck effects

ABSTRACT

In this study, a methodology proposing size reduction of flat steel bearing plates that are commonly used in experimentally evaluating composite deck to transfer and distribute simulated vehicles loads to the top of bridge deck specimens is presented. The proposed methodology considers the tire-to-deck effects based on both a contact pressure distribution model and finite element (FE) simulation results. Ultimate capacity and associated failure modes for the following two loading cases: (i) moment-dominated, and (ii) moment-shear coupled loads were experimentally evaluated. Experimental results showed that the ultimate capacity for moment-dominated loading case is 35.60 kN, while capacity for the same composite deck when subjected to a moment-shear coupled load is 44.34 kN. Experimental results indicated that the initiation of failure for both loading cases was in the form of the development of longitudinal cracks at both top flanges and at outer web section near the loading side. Finite element models that consider lamina damage were developed and analyzed. A good agreement between numerical and experimental results is achieved.

1. Introduction

In the past few decades, the use of fiber reinforced polymer (FRP) composites has grown rapidly in related repair and rehabilitation of existing structures, as well as for all-composite decks and bridges applications [1,2]. In terms of FRP application in bridge engineering, pultruded glass fiber reinforced polymer (GFRP) composites (including web-flange junction research) [1–13] and hybrid GFRP-concrete bridge decks [1,2,14–18] are the preferred choices by structural engineers.

An example of commonly used pultruded GFRP bridge decks is presented in Fig. 1 [12,13]. Typically, composite decks are constructed by bonding individual pultruded GFRP modular hollow profiles using different types of adhesives (refer to Figs. 2 and 3). In general, the pultrusion direction (pull direction) is typically aligned perpendicular to traffic direction. Since late 1990's, numerous experimental and numerical research studies that investigated the global performance of GFRP bridge decks have been reported [7–18]. However, a limited number of publications are available that focus on the local behavior of different elements of pultruded composite decks.

Understanding the local response of tubular GFRP bridge decks at the concentrated tire load position is essential in order to predict the

true behavior of composite bridge decks and to avoid local damage at the contact interface. Traditionally, flat steel plates placed on elastomeric rubber pads are used to simulate tire loads for steel and concrete bridge decks laboratory tests. However, due to both the relatively lower stiffness and shear deformation effects of hollow GFRP bridge decks, much larger local deformation of deck's top flange is expected. It should be noted that due to the higher E_L/G_{LT} ratio of such decks as compared to E/G ratio (E_L is longitudinal modulus of elasticity, G_{LT} is longitudinal shear modulus) for deck made by anisotropic materials, the effects of shear deformation should be considered [19]. As shown in Fig. 4, the contact pressure distribution under a tire load (Fig. 4-a) is with its peak at the middle of the contact length and nonlinearly approaching zero at the ends. However, the contact pressure distribution for laboratory-simulated flat steel plate placed on elastomeric rubber pad generates peaks at the contact zone ends and reduces to zero at the middle of the contact zone (refer to Fig. 4-b) [20–24]. This is because rubber tires are deformable and follow the deflected shape of the deck's top flanges when subjected to actual vehicles' wheel loads, while loads commonly used at the laboratory is conducted by relatively rigid flat steel plates creating a large stiffness mismatch between steel plates and deck top flange. It should be noted that stiffness mismatch between steel and

* Corresponding author.

E-mail address: yql@tongji.edu.cn (Y. Liu).

<https://doi.org/10.1016/j.compstruct.2018.07.082>

Received 18 March 2018; Received in revised form 24 June 2018; Accepted 26 July 2018

Available online 29 July 2018

0263-8223/© 2018 Elsevier Ltd. All rights reserved.

Fig. 1. Pultruded GFRP bridge deck and steel girder system.

Fig. 2. Schematic of pultruded GFRP bridge deck.

Fig. 3. Cross section of pultruded GFRP bridge deck (Units: mm).

GFRP composites (E_{steel}/E_{L-FRP}) is about 10 or more. For this reason, the typical steel plate/elastomeric pad loading detail that is usually employed in concrete and steel decks laboratory evaluation may not be appropriate for GFRP bridge deck laboratory evaluation experiments. The use of steel plate/elastomeric pad loading regime usually lead to a misrepresentation of the actual strength, fatigue behavior, as well as, the ultimate failure mode of the composite deck. In order to address this issue, Zhou et al. [20] examined the response of GFRP deck under two

types of loading: (i) a steel patch with dimensions according to the AASHTO Bridge Design Specifications, and (ii) a simulated tire patch made of an actual truck tire that is reinforced with silicone rubber. Their results showed that the global deflection values under simulated tire loading were much larger than those obtained in the case where steel plate/elastomeric pad loading regime was used. In addition, the failure modes of each loading type were quite different. Majumdar et al. [21] proposed a contact pressure distribution model based on the experimental observation and finite element method (FEA) simulation. Sebastian et al. [22] employed a specified steel plate–rubber pad to generate high shear and low moment to consider the tire-to-deck effect, the results showed that the surfaced specimens' load capacities exceeded those of their unsurfaced counterparts by 90% under shear-dominant loads but exceeded those of their unsurfaced counterparts by 261% under the moment-dominant loads. Sebastian et al. [23] also proposed a curved soffit to simulate tire-to-deck interaction based on FE-predicted deflected profile of the deck's flange and the contact pressure distribution (CPD) are shown to exhibit significant sensitivity to the presence of cracks in the deck, thus rendering the profiled plate–mat system eminently suited to damage detection in decks. In addition, laboratory tests were conducted [24] to investigate tyre-deck contact patch for a certain GFRP bridge deck and empirical definitions of related tyre load is proposed.

Unlike traditional construction materials, GFRP laminates are inhomogeneous in different levels including (i) fiber/resin level, (ii) lamina level, (iii) laminate level, and (iv) structural system level. Details of lamination lay-up of the pultruded GFRP bridge deck profile evaluated in this study are presented in Fig. 5. In designing GFRP composite bridge decks, changes in the geometry will lead to changes in material constituents, lamina thickness, and fiber volume fraction, as well as in material's stiffness and strength. As shown in Fig. 6, one of the known difficulties in accurately analyzing and modeling pultruded composites is that pultruded composites are not precisely laminated structures due to the nature of the pultrusion process [14,15]. For this reason, analytical modeling that considers the nature of composites produced by the pultrusion manufacturing process is required in order to accurately identify such properties. In order to address the aforementioned issues, the elastic engineering constants and ultimate capacity of each lamina are evaluated based on micromechanics in order to provide fundamental input data for the finite element (FE) numerical model.

Fig. 4. Comparison of contact pressure distribution on GFRP deck.

In the paper, a methodology that proposes a size reduction of steel flat plates that are commonly used for transferring loads when testing FRP composite decks is presented. This proposed methodology considering tire-to-deck effect is based on a contact pressure distribution model and utilizes FE simulation results. In this study, the ultimate local capacity and associated failure modes for two cases of loading: (i) moment-dominated, and (ii) moment-shear coupled load is experimentally assessed. Finite element models that consider the failure of

both FRP composites are developed and numerical results are compared to experimental results.

2. Material properties and properties prediction

2.1. Materials properties

The mechanical properties of E-glass fibers and the two-part room-temperature cure epoxy resin are presented in Tables 1 and 2 [25]. The tensile, compressive, in-plane shear, flexural and inter-laminar shear properties of each laminate were obtained from tests according to ISO 527-4 [26], ISO 604 [27], ISO 14129 [28], ISO 14125 [29] and ISO 14130 [30] standards, respectively. Table 3 presents a summary of experimental mechanical properties results obtained for both web and flange laminates.

2.2. Prediction of fiber volume fraction and lamina (Ply) thickness

Unlike advanced composites that are commonly used for aerospace applications with a more accurate lamination lay-up, ply thickness and fiber volume fraction of a pultruded FRP profile are not precise (refer to Fig. 6) and should be evaluated realistically before investigating pultruded composite structures with the multi-scale method. Details for calculating ply thickness, laminate fiber volume fraction and engineering constants of each lamina were reported in previous work by authors [17,18]. The predicted ply thickness and fiber volume fraction of each lamina are summarized in Table 4.

2.3. Prediction of lamina (Ply) stiffness

The engineering constants for each lamina; such as longitudinal modulus E_1 , transverse modulus E_2 , shear modulus G_{12} , and Poisson's ratio ν_{12} could be approximated based on the modified rule of mixture formulae. Table 5 presents ply engineering constants for the composite bridge deck. These values are based on material properties previously presented in Tables 1 and 2, and fiber volume fractions values presented in Table 4. Details for calculating these engineering constants were reported in previous work by the authors [17,18].

2.4. Prediction of lamina (Ply) strength

In order to obtain the lamina strength of a composite laminate, laboratory tests to characterize the strength of the unidirectional and fabric strength are needed. However, several parameters make experimental approach impractical including: (1) since each lamina is orthotropic requiring the identification of several strength parameters; (2) as stated earlier, pultruded composites are not precisely laminated in the true sense due to the nature of the pultrusion process and it is physically impossible to extract test an individual lamina. For these reasons, micromechanics formulations are adopted in this paper in order to provide an initial prediction of the lamina strength. Detailed micro-mechanical equations of the ultimate strength of lamina including longitudinal tensile strength X_T , longitudinal compressive strength X_C , the transverse tensile strength Y_T , the transverse compression strength Y_C and in-plane shear strength S_L were reported in the previous work by authors [31]. The predicted ultimate strengths of both flanges and webs were presented in Table 6. In addition, the fabric layers strengths were treated as unidirectional roving layers with special angles. It should be noted that the purpose of predicted strength is to provide an initial coarse reference in the finite element simulation.

Fig. 5. Laminate lay-up of Pultruded GFRP bridge deck.

Fig. 6. Differences between (a) commercially-produced pultruded composites and (b) aerospace-type laminates.

Table 1
Mechanical properties of E-glass fibers.

Longitudinal modulus (E_{f1})	Transverse modulus (E_{f2})	Poisson's ratio (ν_f)	Shear modulus (G_f)	Tensile strength (X_{ft})	Compressive strength (X_{fc})	Density (ρ)
74.0 GPa	74.0 GPa	0.20	30.80 GPa	2150 MPa	1450 MPa	2560 kg/m ³

Table 2
Mechanical properties of epoxy resin.

Modulus (E_m)	Poisson's ratio (ν_m)	Shear modulus (G_m)	Tensile strength (X_{mt})	Compressive strength (X_{mc})	Shear strength (S_m)	Density (ρ)
3.35 GPa	0.35	1.24 GPa	80 MPa	120 MPa	75 MPa	1160 kg/m ³

3. Load patch determination and experimental programs

3.1. Test setup, instrumentation and procedures

The purpose of evaluating the two load position specimens groups, PL1 and PL2, described earlier, is to investigate the local bearing performance of the composite bridge deck under both moment-dominated

and shear-moment coupled loading patterns. The total span of each specimen is 2000 mm with a width of 512.0 mm. The specimens were simply supported at both ends using two hinges spaced at 1.8 m. For both specimens, 200.0 mm long concrete blocks were cast and placed at the support points to avoid local failure/buckling at these positions (refer to Figs. 8 and 9).

Figs. 8 and 9 show the details of the typical experimental test setup

Table 3
Laminate properties of pultruded GFRP bridge deck.

Item	Top Flange		Bottom Flange		Web	
	Modulus (Gpa)	Strength (Mpa)	Modulus (Gpa)	Strength (Mpa)	Modulus (Gpa)	Strength (Mpa)
Longitudinal Tensile	45.25	503.00	44.73	494.89	44.67	481.95
Transverse Tensile	16.63	60.47	15.23	62.99	22.17	66.00
Longitudinal Compressive	–	615.34	–	576.56	–	547.06
Transverse Compressive	–	188.99	–	194.23	–	200.27
In-plane Shear	6.23	40.15	5.7	50.86	5.97	59.96
Longitudinal Flexural	43.64	1166.98	45.49	1061.36	49.22	1323.43
Transverse Flexural	13.82	108.32	15.11	107.71	19.45	121.12
Inter-laminar shear stress	–	58.46	–	60.89	–	56.88
Longitudinal Poisson's ratio ^a	0.3	–	0.3	–	0.3	–
Transverse Poisson's ratio ^a	0.12	–	0.12	–	0.15	–

Table 4
Predicted lamina thickness and fiber volume fraction of each laminate.

Item		Lamina	Angle	Flange		Web			
				Top	Bottom	Middle	Side	Inner Mortise	Inner Tenon
Roving (mm)	Ply-1	0°	2.26	1.74	1.02	1.77	1.08	2.42
	DBT (mm)	Ply-2	45°	0.13	0.14	0.13	0.13	0.14	0.14
		Ply-3	90°	0.09	0.10	0.09	0.09	0.10	0.09
		Ply-4	–45°	0.13	0.14	0.13	0.13	0.14	0.14
	Roving (mm)	Ply-5	0°	2.26	1.74	1.77	1.77	1.08	2.42
	DBT (mm)	Ply-6	–45°	0.13	0.14	0.13	0.13	0.14	0.14
		Ply-7	90°	0.09	0.10	0.09	0.09	0.10	0.09
		Ply-8	45°	0.13	0.14	0.13	0.13	0.14	0.14
	Roving (mm)	Ply-9	0°	2.26	1.74	1.02	1.77	1.08	2.42
	Total thickness (mm)			7.5	6	4.5	6	4	8
	Fiber volume fraction V_f (%)			64	60	67	67	62	61

Table 5
Predicted engineering constants of each lamina.

Item	Flange		Web			
	Top	Bottom	Middle	Side	Inner Mortise	Inner Tenon
Longitudinal modulus E_1 (GPa)	48.57	45.74	50.69	50.69	47.15	46.45
Transverse modulus E_2 (GPa)	16.06	14.45	17.43	17.43	15.22	14.83
shear modulus G_{12} (GPa)	4.99	4.47	5.46	5.46	4.72	4.59
Poisson's ratio ν_{12}	0.26	0.26	0.25	0.25	0.26	0.26

along with locations of both the laser LVDTs and strain gauges. All specimens were tested using a 500.0 kN capacity calibrated universal testing machine (UTM) and the load was applied under displacement-

control protocol at a constant rate of 1.0 mm/min. Load, deflection and strain signals were continuously recorded during testing using a calibrated high-speed data logger.

Table 6
Predicted ultimate strength of laminas.

Item	Flange		Web			
	Top	Bottom	Middle	Side	Inner Mortise	Inner Tenon
Longitudinal tensile strength, X_T (MPa)	1410.22	1332.42	1491.09	1491.09	1369.98	1330.59
Transverse tensile strength, Y_T (MPa)	43.22	43.75	42.82	42.82	43.49	43.63
Longitudinal compressive strength, X_C (MPa)	951.07	898.61	993.16	993.16	923.94	910.10
Transverse compressive strength, Y_C (MPa)	101.65	100.07	102.98	102.98	100.82	100.41
In-plane shear strength, S_L (MPa)	63.47	62.47	64.31	64.31	62.95	62.68
Transverse shear strength, S_T (MPa)	63.47	62.47	64.31	64.31	62.95	62.68

Fig. 7. Load types (Units: mm).

Fig. 8. Experimental setup and LVDTs layout (Units: mm).

Fig. 9. Strain gages locations and distribution (Units: mm).

3.2. Determination of load patch

The use of FE numerical method to determine load patch size is described in this section. In order to consider the tire-deck contact effects, the steel plate load patch (Fig. 4-c) is reduced in both longitudinal and transverse directions. As is shown in Fig. 7-a, the contact pressure distribution p in Eq. (1) proposed by the ref. [20] is employed to mimic the tire load on GFRP deck. It is assumed that the load under reduced size patch in Fig. 7-b will have the same effect produced by the tire load shown in Fig. 7-a when the top flange deformation under SFP load is close to the CPD load under the same applied load P . It should be noted that this assumption may not be proper when the material damage occurs. All the materials are assumed to act in the elastic stage in order to determine the load patch size considering the quasi-brittle nature of GFRP materials.

$$\frac{p}{p_{ave}} = p_0 + 0.1(x)^6 - 0.14(x)^5 - 1.46(x)^4 + 0.11(x)^3 - 0.29(x)^2 + 0.02(x) \quad (1)$$

where p_0 is the intensity factor defined as maximum pressure divided by average pressure and is assumed to be 1.66 in this work by the Ref. [20], x is the normalized distance defined as path distance divided by half of the total contact length ($-1 \leq x \leq 1$); p_{ave} is average contact pressure and expressed as follows:

$$p_{ave} = \frac{P}{A} = \frac{P}{CP^{0.212}W} \quad (2)$$

where C is empirical constant, P is applied load on single tire, W is the width of tire patch, A is contact area.

Fig. 10. Finite element models of pultruded GFRP deck (Units: mm).

Fig. 11. Iso-surface of top flange displacement.

Fig. 10 shows details of the finite element model used in defining the load patch size in ABAQUS commercial software [32]. In this analysis, continuum shell elements, S4R, with special lamina layup, are used to simulate the pultruded FRP laminates, while the reduced-order solid elements, C3D8R, are used to simulate concrete. The stitched fabric layers, DBT, are simply regarded as three different layers with special angles. The lamina was considered as transversely isotropic materials. Tables 4 and 5 provide summary of lamina thickness, lamina angles, and lamina elastic properties that are calculated based on micro-mechanics approach described earlier in this paper. Based on Refs. [14,15], the concrete elastic modulus and Poisson's ratio used in the analysis are set to be 34.5 GPa and 0.167, respectively. The interactions between FRP laminate and concrete were simulated by a hard contact in the normal direction and a penalty in the tangential direction in order to account for the slip between concrete and the FRP laminate.

As is shown in Fig. 10, the simulation cases were divided into two groups denoted as PL1 and PL2. The load position of group PL1 is located at the mid-span, while the load position of group PL2 is at one-fourth span. As shown, the specimens were simply supported and were spaced at an equal distance of 1800 mm. The original load patch dimensions adopted in the analysis is taken as 200 mm × 300 mm based

on information reported in Ref. [33].

Numerous numerical simulation runs were performed by assuming the common external tangent rectangle of each deformation iso-surface of the top flange as the potential size of load patch (see Fig. 11). Numerical results indicated that the top flange deformation under both load types is similar when the load patch reduced to 11.5% (60 mm × 115 mm) of the original area of 200 mm × 300 mm. The deformation of top flange under CPD load is compared with SFP load with the same applied load P of 0.6 kN in Figs. 12 and 13. A good agreement was achieved between CPD load and SFP load for both PL1 and PL2 specimens. It is noted that the reduced scale of load patch is deck geometry dependent and a closed analytical formulation will be further investigated based on continuum mechanics.

4. Experimental results and discussion

4.1. Load displacement relationship and failure modes

Fig. 14 shows the load-displacement relationship and displacement distributions at the mid-span cross section of PL1 specimen. As shown in Fig. 14-a, initially, the PL1 specimen displacements increased almost

(a) CPD Load

(b) SFP Load

Fig. 12. Top flange displacement comparison of PL1 specimens.

(a) CPD Load

(b) SFP Load

Fig. 13. Top flange displacement comparison of PL2 specimens.

linearly with increasing load with no noticeable stiffness degradation. As shown in Fig. 14-b, the measured displacement, $D1$, near the loading side of PL1 specimen is much larger than measured by $D4$ displacement transducer located at the opposite side of the loading point, indicating the presence of torsional effects. For specimen PL1, the first audible crackling sound was heard at the load of 10.6 kN and a second audible crackling sound followed at a load of 12.6 kN. At a load of 18.0 kN, a longitudinal crack was initiated at the top flanges that are supported by the top of the middle web (refer to Fig. 15-a). Several minor longitudinal cracks were also observed during the load interval between 21.4 kN and 26.0 kN at the outer web near the loading side (refer to Fig. 15-b). At a load level of 35.6 kN, a major crack was formed at the other top flange near the loading point that propagated leading to a sudden load drop. The specimen failed when the longitudinal crack, near the loading point, torn of at an ultimate load of 36.0 kN.

Fig. 16 presents the load-displacement relationship and displacement distributions along the cross section at one-fourth span of PL2

specimen. From the start, the load-displacement curve of PL2 specimen behaved nonlinearly, with some linearity that appeared as the load approached the ultimate load of 44.34 kN. This could be attributed to the fact that the loading position of PL2 specimen is close to roller support, the cast concrete and support roller increase cross-section distortion due to constraint torsion effects by eccentricity local load. The constraint torsion and cross section distortion lead to the displacement uneven. The displacement distribution along the cross section of the PL2 specimen is similar to that of specimen PL1. For specimen PL2, several intermittent crackling sounds were heard as the load increased from 6.5 kN to 11.39 kN. Similar to PL1 specimen, a longitudinal crack (Fig. 17-a) appeared at the top flanges supported by the top of the middle web at the load of 11.65 kN. In addition, several

(a) Load-displacement relationship

(b) Displacements distribution

Fig. 14. Load-displacement relationship and displacement distributions along cross section at mid-span of PL1 specimen.

longitudinal cracks were initiated at the web loading side as the load was increased from 18.0 kN to 22.10 kN. As the load increased from 27.48 kN to 36.43 kN, additional longitudinal cracks were formed and propagated as the load increased from 27.48 kN to 36.43 kN. At a load of 32.0 kN, a longitudinal crack, near the loading point, was observed and propagated rapidly. Also, several cracks were developed at the webs that propagated rapidly as the applied load approached 42.64 kN, leading to an abrupt strength degradation.

4.2. Strain distribution and load-strain relationship

Under localized load, the deformation of a pultruded GFRP bridge deck may be regarded as three superimposed modes. The first mode is the three-point bending behavior where the top flange is subjected to compressive stresses, while the bottom flange is subjected to tensile stresses. The second mode is in the form of a local transverse deformation of continuous plates that are supported by three different

semi-rigid web-flange junctions elastic supports. The third mode is the local cross section distortion caused by the complex loading status. It is noted that the effect of the third mode is relatively small before the load increased to the ultimate capacity.

As is shown in Figs. 18 and 19, the predicted overall strain distributions along both sides of the webs' height agreed well with the experimental strain distributions obtained from the three-point bending beam tests, specifically; the compressive strains above the neutral axis and tensile strains below neutral axis. However, one can notice that both strain distributions along the web heights behaved nonlinearly that violated the Bernoulli's hypothesis due to local transverse deformation effects. The neutral axis of web tenon (refer to Fig. 2) moved closer towards the bottom flanges, while for the web mortise, the neutral axis moved towards the top flanges. This deformation pattern is attributed to the coupling of the bending and torsional moments at the middle span cross-section that is subjected to an eccentric local load.

The strain distributions along the top flange cross-section of both

(a) Top flange failure mode

(b) Web failure near the loading side

Fig. 15. Failure modes comparison of PL1 specimens.

PL1 and PL2 specimens are presented in Figs. 20 and 21. As shown in Figs. 20-a and 21-a, the longitudinal strains at the top flanges along the cross section of both PL1 and PL2 specimens were subjected to compressive stresses. From these figures, one can see that the top flange longitudinal strains, closer to the loading points, were larger than other locations as the load increased from 0.0 kN to 16.0 kN, while the strains gradually reduced starting from the web tenor to web mortise as the load increased from a load of 16.0 kN to the specimen's ultimate load. This resulted in the development of multiple cracks that appeared at the web tenor and were accompanied by stiffness degradation that led to a much larger value of flange compressive strain at the web tenor top side (refer to Fig. 2). As presented in Figs. 20-b and 21-b, the top flanges transverse strains along both PL1 and PL2 specimens cross-section were in tensile mode. Also, results indicated that the measured strains at the flange at the top of the middle web are much larger than those at other

positions due to the middle web constraint that has a high potential of developing such tensile cracks at this region.

Figs. 22 and 23 present the strain distributions along the bottom flange cross section for both PL1 and PL2 specimens. As shown in Figs. 22-a and 23-a, tensile longitudinal strains were developed along the bottom flanges cross-sections of both PL1 and PL2 specimens. From these figures, one can notice that the longitudinal strains along bottom flange cross-section, at the loading side cell, are much larger than those developed at the opposite cell and that the bottom flange longitudinal strains near the bottom of web tenor are the largest. As shown in Figs. 22-b and 23-b, compressive transverse strains were developed at the bottom flanges located at the loading cell, while tensile strains were observed at the opposite cell. The compressive transverse strain at the loading cell is caused by the Poisson effects that are opposite to the bottom flange longitudinal strains.

Fig. 16. Load-displacement relationship and displacement distributions along cross section at one-fourth span of PL2 specimen.

Figs. 24 and 25 show the strain distributions along the top flange span of PL1 and PL2 specimens. As shown in these figures, tensile longitudinal strains were developed at the top flanges, while the transverse compressive strains were captured at top flanges of both specimens PL1 and PL2. This can be attributed to the fact that the top flange is locally stretched in the longitudinal direction to adapt to the top flange large local deformation, while it is transversely compressed due to the Poisson effects. As expected, both longitudinal and transverse strains along the span showed larger values at locations close to the loading points.

Strain distributions along the span of the bottom flanges of specimens PL1 and PL2 are presented in Figs. 26 and 27. Tensile longitudinal strains were generated at the bottom flanges due to the three-point bending effects, while compressive transverse strains were developed at the bottom flanges due to the Poisson effects.

5. Numerical simulations

In this section, the finite element model used in determining the size of the load patch (refer to Fig. 10) is extended to include the damage

prediction of both FRP and concrete. The interactions between FRP plates and concrete were simulated by hard contact in the normal direction and penalty in the tangential direction in order to consider slip between concrete and FRP. The friction coefficient with 0.7 is used based on Ref. [14]. In the following sections, numerical simulation results are compared with experimental results.

5.1. FRP damage model

Due to the fact that no obvious delamination damage was observed in the experimental program, it was decided to utilize the Hashin's theory [34] as the initiation failure criteria in the analysis. This criterion consists four different damage initiation mechanisms, namely; (i) fiber tension (f_t) mechanism, (ii) fiber compression (f_c) mechanism, (iii) matrix tension (m_t) mechanism, and (iv) matrix compression (m_c) mechanism as expressed in Eqs. (3)–(6). Damage initiation properties of FRP lamina were calculated based on micromechanics and are presented in Table 6.

Fiber Tension ($\sigma_{11} \geq 0$):

(a) Top flange failure mode

(b) Web failure near the loading side

Fig. 17. Failure modes comparison of PL2 specimens.

$$F_{ft} = \left(\frac{\sigma_{11}}{X_T} \right)^2 + \left(\frac{\sigma_{12}}{S_L} \right)^2 = 1 \quad (3)$$

Fiber Compressive ($\sigma_{11} < 0$):

$$F_{fc} = \left(\frac{\sigma_{11}}{X_C} \right)^2 = 1 \quad (4)$$

Matrix Tension ($\sigma_{22} \geq 0$):

$$F_{mt} = \left(\frac{\sigma_{22}}{Y_T} \right)^2 + \left(\frac{\sigma_{12}}{S_L} \right)^2 \quad (5)$$

Matrix Compressive ($\sigma_{22} < 0$):

$$F_{mc} = \left(\frac{\sigma_{22}}{2S_T} \right)^2 + \left[\left(\frac{Y_C}{2S_T} \right) - 1 \right] \frac{\sigma_{22}}{Y_C} + \left(\frac{\sigma_{12}}{S_L} \right)^2 \quad (6)$$

where σ_{ij} are the components of the effective stress tensor, X_T is the longitudinal tensile strength, X_C is the longitudinal compressive

strength, Y_T is the transverse tensile strength, Y_C is the transverse compressive strength, S_L and S_T are the longitudinal and transverse shear strengths, respectively.

Once a damage initiation criterion is satisfied, further loading will cause degradation of the material stiffness coefficients. The reduction of the stiffness coefficients is controlled by damage variables that assume values between zero (undamaged state) and one (full-damage state). The damage variable for each failure mode [35] is expressed as follows:

$$d_I = \frac{\delta_{I,eq}^f (\delta_{I,eq}^0 - \delta_{I,eq}^0)}{\delta_{I,eq}^f (\delta_{I,eq}^f - \delta_{I,eq}^0)} \quad \delta_{I,eq}^0 \leq \delta_{I,eq} \leq \delta_{I,eq}^f \quad (7)$$

where $\delta_{I,eq}^0$ is the equivalent displacement at which the initiation criterion is satisfied, $\delta_{I,eq}$ is the equivalent displacement, and $\delta_{I,eq}^f$ is the equivalent displacement at which the materials is fully damaged. The expressions of $\delta_{I,eq}^0$, $\delta_{I,eq}$ and $\delta_{I,eq}^f$ are described in Refs. [32,35]. The fully-damaged equivalent displacement, $\delta_{I,eq}^f$, is typically calculated

(a) Loading side

(b) Opposite to loading side

Fig. 18. Strain distributions along web of PL1 specimen.

(a) Loading side

(b) Opposite to loading side

Fig. 19. Strain distributions along web of PL2 specimen.

based on the fracture energy, $G_{I,c}$, and is calculated by Eq. (8) in terms of the bilinear-softening model. Table 7 presents values of fracture energy that are used in this paper which are based on procedures described in Ref. [32]. It should be noted that the fracture energies need to be further investigated by fracture energy tests (including typical I and II fracture tests). In addition, the fracture energies have small effects on the ultimate capacity simulation; however, these energies have larger effects on post-failure behavior. It should also be noted that the post-failure (collapse) behavior is not the main focus of this study. However, this issue is being investigated and will be reported in future.

The following expression describes the fully-damaged equivalent displacement, $\delta_{I,eq}^f$:

$$\delta_{I,eq}^f = \frac{2G_{I,c}}{\sigma_{I,eq}^0} \quad (8)$$

where $\sigma_{I,eq}^0$ is the equivalent stress at which an initiation criterion is satisfied. The expression for determining the equivalent stress, $\sigma_{I,eq}^0$, is described in Refs. [32,35].

It is well known that material models that exhibit softening behavior and stiffness degradation always face serious convergence

difficulties in implicit FE analysis. For this reason, it is common to adopt a viscous regulation scheme in order to alleviate such convergence difficulties. The viscous regulation scheme used in ABAQUS/Standard is expressed in Eq. (9) [32,35]. In this study, the viscosity coefficient, η_I , of each failure mode is assumed to be 0.0005 s.

$$\dot{d}_I^v = \frac{1}{\eta_I} (d_I - d_I^v) \quad (9)$$

where η_I is a viscosity coefficient representing time relaxation of the viscous system, and d_I^v is the regularized damage variable for failure mode I.

5.2. Concrete damage model

In this study, the concrete plastic-damage model was adopted that considers concrete cracking and crushing. The uniaxial compressive stress-strain relationship [14] was obtained using Eqs. (10)–(16), while

(a) Longitudinal strain

(b) Transverse strain

Fig. 20. Strain distributions along cross section of top flange of PL1 specimen.

the tensile stress-strain relationship was assumed to be linear prior to reaching the ultimate tensile stress that is followed by a softening behavior as described in CEB-FIP [36,37]. The plastic property is defined as the difference between *total* property, calculated by Eqs. (10)–(16), and the corresponding concrete *elastic* property. Same parameters used in the previous work [14] are employed in the current study, where both the elastic modulus and the Poisson's ratio values identical to those described in Section 3 of this paper. In this analysis, the uniaxial compressive strength is 37.5 MPa, the uniaxial tensile strength is 3.28 MPa, the dilation angle is 30°, the flow potential of skewness is 0.1, the ratio of initial equi-biaxial compressive yield stress to initial uniaxial compressive stress f_{bo}/f_{co} is 1.16, the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield is 0.67, and the coefficient of viscosity is 0.0005. The

parameters of concrete plastic-damage model are summarized in Table 8.

(a) Longitudinal strain

(b) Transverse strain

Fig. 21. Strain distributions along cross section of top flange of PL2 specimen.

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k-2)\eta} \quad (10)$$

$$\eta = \varepsilon_c / \varepsilon_{c1} \quad (11)$$

$$k = 1.05 E_{cm} \varepsilon_{c1} / f_{cm} \quad (12)$$

$$\frac{\sigma_t}{f_t} = f(w) - \frac{w}{w_c} f(w_c) \quad (13)$$

$$f(w) = \left[1 + \left(\frac{c_1 w}{w_c} \right)^3 \right] e^{\left(-\frac{c_2 w}{w_c} \right)} \quad (14)$$

$$G_f = 73 f_{cm}^{0.18} \quad (15)$$

$$w_c = \frac{5.14 G_f}{f_t} \quad (16)$$

where σ_c is the compressive stress, ε_c is the compressive strain, σ_t is the

tensile stress, ε_{cl} is the peak strain, η is the ratio between the com-

However, and as shown in Fig. 29, the predicted ultimate capacity of specimen PL2 is slightly lower than the capacity obtained from the test.

Fig. 22. Strain distributions along section of bottom flange of PL1 specimen.

pressive and peak strains, f_{cm} is the ultimate compressive strength, E_{cm} is the elastic modulus, f_t is ultimate tensile stress, w is crack opening displacement, w_c is crack opening displacement at which stress can no longer be transferred, $c_1 = 3.0$, $c_2 = 6.93$ for normal density concrete, and G_f is the energy required to open a unit crack area.

5.3. Comparison between numerical and experimental results

Figs. 16 and 17 present a comparison between experimental and numerically-simulated failure modes. The FE results showed that damages were initiated at both the top flange, and the web that led to the ultimate failure of the two specimens evaluated in this study. As shown in these figures, numerical results for fully-damaged area agreed well with the cracks initiation observed in the laboratory experiments.

Comparisons between experimental and numerical load-displacement and load-strain relationships for specimens PL1 and PL2 are presented in Figs. 28 and 29, respectively. As shown in Fig. 28, experimental results obtained from specimen PL1 agreed well with the corresponding numerical displacements, strains, and ultimate capacity.

Fig. 23. Strain distributions along section of bottom flange of PL2 specimen.

In addition, a small difference between load-displacement curves and load-strain curves is observed. Also, one can see from these figures that as the load increased, approaching specimens ultimate capacity, a decrease in bottom flange displacement, at the loading position, is observed (stage PQ in Fig. 28-a, and stage MN in Fig. 29-a). The decrease in local displacement was observed in both experimental and FEM numerical results, which is quite different from the corresponding results previously obtained for a general bridge deck with solid sections such as the hybrid GFRP-concrete bridge deck described in Refs. [14,15]. This can be attributed to the major cross-sectional distortion. Fig. 30 presents an exaggerated sketch of the distorted cross-section at the loading zone based on FEA simulation results. As shown in Fig. 30, the lateral displacement components U_A and U_B led to the development

(a) Longitudinal strain

(b) Transverse strain

Fig. 24. Strain distributions along span of top flange of PL1 specimen.

(a) Longitudinal strain

(b) Transverse strain

Fig. 25. Strain distributions along span of top flange of PL2 specimen.

of rotational moments M_C and M_D , which will make the uplift displacement at D2 position. As stated earlier, the deformation of a pultruded GFRP bridge deck can be described by three superimposable modes: (i) three-point bending, (ii) local transverse deformation, and (iii) cross-sectional distortion. Once the ultimate capacity is reached, the effects of the first two modes (i.e. three-point bending, and local transverse deformation) gradually diminish, while the cross-sectional distortion becomes the predominant mode resulting in top flange displacement increase. In addition, the sudden stiffness degradation of the load-displacement curve for PL2 specimen at point E (see Fig. 29-a) can be attributed to the occurrence of local cross-sectional distortion. As is shown in Fig. 31, de-bonding between the composite deck and the concrete was initiated due to transverse torsion that occurred at point E (refer to Fig. 29-a). The effect of local cross-sectional distortion of the GFRP bridge deck gradually increased as the debonding took place resulting in an increase of displacement D2. For GFRP bridge decks, the cross-sectional distortion has a larger impact on its behavior due to several reasons including: (i) the hollow profile inherent characteristics; (ii) the relatively low longitudinal elastic modulus of GFRP pultrude

composites, and (iii) the smaller thickness of the flanges and webs. For above reasons, much attention should be paid to these distortion contributing factors when designing GFRP thin-walled cellular bridge decks. Results of this study highlighted the importance of the inclusion of inclined webs in order to alleviate this cross-sectional distortion.

6. Conclusions

This paper presents a summary of results of an experimental and numerical study that focuses on assessing the local bearing behavior of a composite deck. In this study, the inherent characteristics of a typical modular pultruded composite bridge deck are considered. Finite element models were developed and analyzed for different deck components that considered both lamina and concrete damage. Failure modes observed from the tests were compared with those numerically simulated and agreement between results was achieved. Based on the results of this study, the following conclusions can be drawn:

(1) A methodology for reducing the size of the steel flat plate that is

Fig. 26. Strain distributions along span of bottom flange of PL1 specimen.

commonly used in experimental evaluation of composite decks is proposed. The proposed methodology considers the tire-to-deck effects based on a contact pressure distribution model, and FE simulation results. The top flange deformation for the two loading patterns evaluated in this study found to be much closer when the load patch is reduced to 11.5% (60 mm × 115 mm) of original area (200 mm × 300 mm).

(2) Experimental results indicated that the composite deck ultimate capacity is 35.6 kN for the moment-dominated loading case, while it is 44.34 kN for the moment-shear coupled loading case. Longitudinal cracks developed at the top flanges and at outer webs, near the loading side, led to the final failure of the deck specimens.

Fig. 27. Strain distributions along span of bottom flange of PL2 specimen.

Table 7
Fracture energies of fiber-reinforced epoxy.

$G_{ft,c}$ (N/mm)	$G_{fc,c}$ (N/mm)	$G_{mt,c}$ (N/mm)	$G_{mc,c}$ (N/mm)
12.5	12.5	1.0	1.0

(3) Finite element models that consider laminate failure were developed and results were compared to local bearing experimental results. A good agreement between numerical and experimental results for specimen PL1 was achieved, however, the numerical simulated ultimate capacity of specimen PL2 was lower than that was obtained from the tests.

Table 8
The parameters of plastic-damage model.

Dilatation angle	Flow potential of skewness	f_{b0}/f_{c0}	k	coefficient of viscosity
30	0.1	1.16	0.6667	0.0005

Notes: The expression “ f_{b0}/f_{c0} ” is the ratio of initial equi-biaxial compressive yield stress to initial uniaxial compressive yield stress, and the letter ‘ k ’ represents the ratio of the second stress invariant of the tensile meridian to that on the compressive meridian at initial yield for any given value of the pressure invariant.

(a) Load-displacement relationship

(b) Load-strain relationship

Fig. 28. Displacement and strain comparison of PL1 specimens.

(a) Load-displacement relationship

(b) Load-strain relationship

Fig. 29. Displacement and strain comparison of PL2 specimens.

Fig. 30. Cross-Section distortion at the loading position.

Fig. 31. Debonding of Concrete and GFRP deck of PL2 specimen.

Acknowledgements

This research study was funded by National Natural Science Foundation [Grants # 51578406] of the People's Republic of China.

References

- [1] Mosallam AS, Bayraktar A, Elmikawi M, Pul S, Adanur S. Polymer composites in construction: an overview. *SOJ Mater Sci Eng* 2015;2(1):25. (Open Access).
- [2] Bakis C, Bank LC, Brown V, Cosenza E, Davalos J, Lesko J, et al. Fiber-reinforced polymer composites for construction-state-of-the-art review. *J Compos Constr* 2002;6(2):73–87.
- [3] Kahya V, Mosallam A. Dynamic analysis of composite sandwich beams under moving mass. Kahrmanmaras Sutcu Imam University (KSU). *J Eng Sci* 2011;14(1):18–25.
- [4] Mosallam AS, Haroun MA, Kreiner J, Dumlaio C, Abdi F. Structural evaluation of all-composite deck for Schuyler Heim bridge. In: Proceedings of the SAMPE 2002 Conference, 2002, Long Beach, CA, May 12–16, 2002.
- [5] Keller T, Rothe J, De-Castro J, Osei-Antwi M. GFRP-balsa sandwich bridge deck: concept, design, and experimental validation. *J Compos Constr* 2013;18(2):04013043.
- [6] Sahirman S. Fiber reinforced polymer (FRP) bridge deck life-cycle cost analysis. West Virginia University; 2009. Ph.D. Thesis.
- [7] Zi G, Kim BM, Hwang YK, Lee YH. The static behavior of a modular foam-filled GFRP bridge deck with a strong web-flange joint. *Compos Struct* 2008;85(2):155–63.
- [8] Xin H, Mosallam A, Liu Y, Yang F, Zhang Y. Hygrothermal aging effects on shear behavior of pultruded FRP composite web-flange junctions in bridge application. *Compos. Part B: Eng.* 2017;110:213–28.
- [9] Lee J, Kim Y, Jung J, Kosmatka J. Experimental characterization of a pultruded GFRP bridge deck for light-weight vehicles. *Compos Struct* 2007;80(1):141–51.
- [10] Xin H, Mosallam A, Liu Y, Wang C, Zhang Y. Impact of hygrothermal aging on rotational behavior of web-flange junctions of structural pultruded composite members for bridge applications. *Compos. Part B: Eng.* 2017;110:279–97.
- [11] Xin H, Mosallam A, Liu Y, Xiao Y, He J, Wang C, et al. Experimental and numerical investigation on in-plane compression and shear performance of a pultruded GFRP composite bridge deck. *Compos Struct* 2017;180:914–32.
- [12] Xin H, Mosallam A, Liu Y, Wang C, Zhang Y. Analytical and experimental evaluation of flexural behavior of FRP pultruded composite profiles for bridge deck structural design. *Constr Build Mater* 2017;150:123–49.
- [13] Salim HA, Barker M, Davalos JF. Approximate series solution for analysis of FRP composite highway bridges. *J Compos Constr* 2006;V10(4):357–66.
- [14] Xin H, Liu Y, He J, Fan H, Zhang Y. Fatigue behavior of hybrid GFRP-concrete bridge decks under sagging moment. *Steel Compos Struct* 2015;18:925–46.
- [15] Xin H, Liu Y, Du A. Thermal analysis on composite girder with hybrid GFRP-concrete deck. *Steel Compos Struct* 2015;19(5):1221–36.
- [16] He J, Liu Y, Chen A, Dai L. Experimental investigation of movable hybrid GFRP and concrete bridge deck. *Constr Build Mater* 2012;26(1):49–64.
- [17] Sebastian W, Gegeshidze G, Luke S. Positive and negative moment behaviours of hybrid members comprising cellular GFRP bridge decking epoxy-bonded to reinforced concrete beams. *Compos B Eng* 2013;45(1):486–96.
- [18] Zuo Y, Mosallam A, Xin H, Liu Y, He J. Flexural performance of a hybrid GFRP-concrete bridge deck with composite T-shaped perforated rib connectors. *Compos Struct* 2018;194:263–78.
- [19] Mosallam AS, Bank LC. Short-term behavior of pultruded fiber-reinforced plastic frame. *J Struct Eng* 1992;118(7):1937–54.
- [20] Zhou A, Coleman JT, Temeles AB, Lesko JJ, Cousins TE. Laboratory and field performance of cellular fiber-reinforced polymer composite bridge deck systems. *J Compos Constr* 2005;9(5):458–67.
- [21] Majumdar PK, Lesko JJ, Cousins TE, Liu Z. Conformable tire patch loading for FRP composite bridge deck. *J Compos Constr* 2009;13(6):575–81.
- [22] Sebastian WM, Keller T, Ross J. Influences of polymer concrete surfacing and local load distribution on behaviour up to failure of an orthotropic FRP bridge deck. *Compos B Eng* 2013;45(1):1234–50.
- [23] Sebastian WM, Webster T, Kennedy C, Ross J. Profiled metal plate-cork mat loading systems on cellular FRP bridge decks to reproduce tire-to-deck contact pressure distributions. *Constr Build Mater* 2013;49:1064–82.
- [24] Sebastian WM, Ralph M, Poulton M, Goacher J. Lab and field studies into effectiveness of flat steel plate-rubber pad systems as tyre substitutes for local loading of cellular GFRP bridge decking. *Compos B Eng* 2017 September;15(125):100–22.
- [25] Soden PD, Hinton MJ, Kaddour AS. Lamina properties, lay-up configurations and loading conditions for a range of fibre-reinforced composite laminates. *Compos Sci Technol* 1998;58(7):1011–22.
- [26] ISO, B. 527-4. Plastics—determination of tensile properties—part 4: test conditions for isotropic and orthotropic fibre-reinforced plastic composites. International Organization for Standardization (ISO), 1997, Geneva, Switzerland.
- [27] ISO, B. 604. Plastics—determination of compressive properties. International Organization for Standardization (ISO), 2002, Geneva, Switzerland.
- [28] ISO, B. 14129. Fibre-reinforced plastic composites—Determination of the in-plane shear stress/shear strain response, including the in-plane shear modulus and strength, by the ± 45 tension test method. International Organization for Standardization (ISO), 1997, Geneva, Switzerland.
- [29] ISO, B. 14125. Fiber-reinforced plastic composites – Determination of flexural properties. International Organization for Standardization (ISO), 1998, Geneva, Switzerland.
- [30] ISO, B. 14130. Fibre-reinforced plastic composites—Determination of apparent interlaminar shear strength by short-beam method, International Organization for Standardization (ISO), 1998, Geneva, Switzerland.
- [31] Xin H, Liu Y, Mosallam A, et al. Evaluation on material behaviors of pultruded glass fiber reinforced polymer (GFRP) laminates. *Compos Struct* 2017;182:283–300.
- [32] ABAQUS V. 6.14 Documentation. Dassault Systems Simulia Corporation; 2014.
- [33] P.R. China General Code for Design of Highway Bridges and Culverts, Ministry of Communications of China, Beijing: China Communication Press, 2004. (In Chinese).
- [34] Hashin Z. Failure criteria for unidirectional fiber composites. *J Appl Mech* 1980;47(2):329–34.
- [35] Lapczyk I, Hurtado JA. Progressive damage modeling in fiber-reinforced materials. *Compos A Appl Sci Manuf* 2007;38(11):2333–41.
- [36] Eurocode 4, Design of composite steel and concrete structures; Part 1-1: General rules and rules for buildings, 1994.
- [37] FIP Bulletin 55, International federation for structural concrete (CEB-FIP). : Model Code 2010, 2010, First complete draft-Volume 1, Switzerland.