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Efficient Use of CFRP Stay-in-Place Form for Durable Concrete Bridge Decks

by L. Cheng and V.M. Karbhari

Synopsis: This paper presents the development of a steel-free concrete bridge deck reinforced with carbon fiber-reinforced polymer (CFRP) stay-in-place (SIP) form. The SIP form has a configuration of a flat laminated CFRP plate stiffened with rectangular stand-ups filled with nonstructural foam and interlocking ribs at the interface. Thin layers of CFRP mesh are used for top tensile reinforcement at intermediate continuity regions. Performance evaluation of short-term static flexure was conducted through tests on a series of 610 mm (2 ft) wide deck specimens. Dynamic response of the system (for example, natural frequencies and mode shapes) was characterized using a forced vibration testing method. Furthermore, long-term behavior under fatigue simulating traffic loads was experimentally assessed using a full-scale continuously spanned specimen. The observations from these laboratory tests on load-carrying capacity and failure modes showed a satisfactory and efficient design of the system. These test results were further used to calibrate a finite-element based nonlinear model (ABAQUS) for numerical simulation and development of a simplified design procedure. Environmental effects due to temperature, creep, and shrinkage were considered using the calibrated numerical model, the results of which showed insignificant residual stress caused by these effects between concrete and CFRP composites over time.

Keywords: bridge deck; fiber-reinforced polymer (FRP); stay-in-place (SIP); steel-free

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INTRODUCTION

Concrete bridge decks have been the most deficient structure among all bridge components primarily due to the corrosion associated with their steel reinforcement[1]. Statistics showed that U.S. bridges last 68 years on average but their decks last 35 years, only about half of their superstructure[2]. The corrosion in bridge decks can be caused by factors such as deicing agents and salt water penetrating the porous concrete attacking the reinforcing steel, and subsequently causing spalling of concrete. Conventional protective methods include the replacement of steel reinforcement with epoxy coated galvanized or stainless steel bars, and treating the concrete surface with siloxanes or cathodic protection, etc. These methods are very costly and of limited use, and oftentimes not enough to prevent the inevitable penetration of salts into the concrete slab. It is therefore imperative to build deck systems that themselves have longer durability and require less maintenance during the service lifetime of the bridge. A potential solution to this challenge has been the use of new materials or through implementation of new structural systems. Fiber Reinforced Polymer (FRP) composites offer an attractive possibility to achieve both, and have had substantial advancement in the recent years in the civil engineering community.

FRP composites have been used in many bridge deck applications due to their light weight, improved corrosion resistance, better long-term durability, and potentially low maintenance and life-cycle costs than conventional materials, such as steel and concrete. They have primarily been used to replace the corrosion-prone steel reinforcement in forms of rebars or tendons[3-4], 2-dimensional and 3-dimensional gratings and grids[5]. FRP continuous plates have also been used for durability considerations[6] and more recently for both tensile reinforcement[7-8] in replacement of steel bars and Stay-In-Place (SIP) permanent form. This hybrid concept combining low-cost but high compressive strength concrete material with high performance FRP composites appears to be very cost effective. The structural performance of many of these systems has been investigated through numerous laboratory and field tests. The following general observations are made based on the current state-of-the-art[9]: a) The design of most concrete deck systems utilizing FRP composites are primarily driven by the flexural-shear strength of the concrete slab; b) In most of these systems, punching shear and fatigue state do not appear to be the governing limit states for the design; c) Compared to the conventional reinforced concrete (RC) decks, hybrid FRP-concrete decks generally display higher durability with less deterioration in stiffness under design truck loads; d) It is commonly recommended to over-reinforce FRP composites to avoid sudden brittle type of failure in FRP composites while forcing the crushing type of failure in concrete; and e) catastrophic failure is not common in hybrid FRP-concrete deck systems.

This paper presents research work that demonstrates the feasibility of an innovative hybrid deck system that utilizes Carbon Fiber Reinforced Polymer (CFRP) Stay-In-Place (SIP) permanent form as tensile reinforcement for a concrete slab that is free of steel. The SIP form has a configuration of a flat laminated CFRP plate stiffened with rectangular stand-ups filled with non-structural foam and interlocking ribs at the interface. The utilization of this CFRP SIP form in the deck can not only accelerate the construction process, but enhance the durability of the bridge Aspects including the conceptual design, laboratory short-term static response, long-term behavior due to environment and fatigue effects, and characterization of system's dynamic features, are extensively discussed in this paper. A simplified design procedure with explicit design equations is discussed in details elsewhere[10]; therefore is excluded in this paper.

RESEARCH SIGNIFICANCE

The application of FRP composites as structural formwork for concrete structures is relatively new when compared to their other infrastructural applications. Research reported in this paper demonstrates the feasibility of using CFRP based open form and flexural reinforcement for concrete bridge decks through extensive laboratory and numerical investigations on their short-term and long-term structural behavior. Results derived from this work provide performance data to the current knowledge database, illustrating an economical alternative for a durable decking system for rapid bridge construction. The results are also made useful to practitioners by providing a simplified design method conforming to the current concrete design practice.

DESIGN CONCEPT AND MATERIALS

Hybrid FRP-concrete deck system

Due to differences in material characteristics, the application of FRP composites to bridge systems requires critical consideration to ensure development of suitable design concepts that facilitate the efficient use of materials, yet meet all the requirements of the structure and the system. To minimize the quantity of costly fiber and resin materials, a hybrid concept of FRP and concrete is implemented in the current design, where the hybrid deck system consists of a steel-free concrete slab cast on top of CFRP deck panel (as shown in Figure 1). The stiffened deck panel acts as both structural formwork as well as the flexural reinforcement for the concrete slab. Primary design considerations of the deck system include: (a) the deck panel by itself shall carry the construction load, i.e., the weight of the fresh concrete, 4.8 kN/m^2 (100 lb/ft^2), and construction labors without excessive deflection; (b) the deck panel, when cast with a 203 mm (8 in) thick concrete slab, shall satisfy the desired strength and serviceability requirements; and (c) in order to avoid brittle failure in the FRP reinforcement, the entire deck section shall be over-reinforced with respect to a balanced design at ultimate for flexure, forcing a concrete crushing type of failure mode. This implies that the FRP reinforcement will have relatively low stress levels under normal loads and strength reserve. Short fibrillated polypropylene fibers, which are chemically inert and hydrophobic[11], are specially added during the mixing of concrete for crack control in concrete due to shrinkage effect. A shear rib-sand type of interface is incorporated on the top surface of the CFRP panel to ensure an appropriate level of force transfer between the concrete and the deck panel. A typical design of multi-span bridges involves regions where concrete section experiences negative bending moment under traffic loads. Tensile reinforcement in the form of carbon/epoxy mesh is thus provided near the top surface of the slab.

Materials

The composite deck panel, as shown in Figure 2, consists of a bottom plate that is 2.254 m (7.4 ft) long and 6.3 mm (0.248 in) thick with end hooks for potential connections to girders[9] and adhesively bonded rectangular stiffeners filled with foam. The bottom plate is composed of 8 layers of unidirectional carbon fabric (designated as C) with an areal weight of 305 g/m^2 (i.e., 9 oz/yd^2, the unit thickness of each layer is 0.38 mm or 0.015 in) and 4 layers of E-glass chopped strand mat (designated as E) with an areal weight of 458 g/m^2 (i.e., 13.5 oz/yd^2, unit thickness of 0.92 mm or 0.036 in), in a symmetric lay-up scheme of $[C/E/C_2/E/C]_S$ (Mark 4). The rectangular stiffeners that are 41 mm (1.614 in) wide and 105 mm (4.134 in) high with a spacing of 305 mm (12 in) contain primarily unidirectional carbon fabric and foam core and are adhesively bonded onto the bottom plate. The stiffeners are designed to provide the required stiffness for construction loads and assist load transferring from concrete slab to the bottom plate. Alternative design schemes using I-beam shaped stiffeners and corrugated deck panel were considered at the conceptual design stage, as shown in Table 1

Table 1. For the I-beam design, the bottom plate was kept the same as the original design (Mark 4) and both the web and flange were designed using Mark 2 (same as in the original scheme). The corrugated design used Mark 4 uniformly in the panel. To provide the equivalent level of flexural stiffness for construction load in the deck, the design using I-beam shaped stiffeners (with the same depth and spacing as the rectangular one but a slightly larger width, 41×105 mm) and the corrugated design (with 2 corrugations of the size of 41×74 mm with the same spacing) can possibly be used to replace the rectangular design (41×105 mm). Since similar amount of carbon fiber reinforcement will be needed for all options (100%, 97% and 106% for rectangular, I-beam and corrugated design), the original panel design using rectangular stiffeners was adopted as the prototype in this research. To enhance the shear interaction between the concrete slab and the deck panel, the top surface of the entire panel is sand treated and additionally installed with shear ribs made of sand-epoxy paste. The longitudinal and transverse modulus of the bottom reinforcing plate (Mark 4) is approximately 60.4 GPa (8765 ksi) and 6.9 GPa (995 ksi), respectively, according to ASTM D3039-76[12] testing method. The tensile reinforcement provided by this CFRP plate is equivalent to 2 mild steel bars, 20M with the unit area of 300 mm^2 (0.47 in^2) and the modulus of 204 GPa (29.6 msi) which result in an equivalent axial tensile stiffness (*EA*) of 112 MPa-m^2 (25.2 msi-in^2) in the reinforcement. A comparative study with other deck systems (e.g., conventional RC deck, metal deck-concrete system, and GFRP bar

reinforced deck) is in progress with the author. The total depth of the concrete slab is selected as 200 mm (8 in), compatible with the typical steel reinforced concrete bridge slabs per AASHTO[13].

Fibrillated high performance polypropylene fibers (S-152 HP) were used in the concrete mixing (Figure 2b). It has a nominal length of 50.8 mm (2 in) and an elastic modulus of 3500 MPa (500 ksi). The specific gravity of the fiber is 0.91 and the ultimate elongation is 15%. A fiber volume content of 0.88% is selected for the concrete mix based on the construction practice (Canadian Highway Bridge Design Code, CHBDC[14]). Carbon/epoxy composite mesh (Figure 2c) made of longitudinal and transverse AS4 strands (Hexel, spaced at 25 mm or 1 in) is used in the continuity region (negative moment region), being placed about 25 mm (1 in) below the concrete top surface. The tensile strength of the mesh is experimentally obtained as 841.7 MPa (122 ksi) from a series of 254 mm (10 in) long sample strands. The amount of fiber mesh that is needed is determined based on the tensile stress requirement at the continuity regions. Normal weight concrete was used with a maximum aggregate size of 12.7 mm (1/2 in) and an average compressive strength of 44.6 MPa (6.5 ksi), based on the standard cylinder tests on the same day of testing.

SHORT-TERM STRUCTURAL BEHAVIOR

Static flexure and shear

Eight specimens (SF1–5 and SB1–3, Table 2) were designed to investigate the flexural and interfacial response between the concrete slab and the CFRP panel, emphasizing on the effect of the spacing of the rectangular stiffeners and interfacial shear ribs. In order to study the behavior of the FRP-concrete deck itself, the effect of mixing fibers in concrete was excluded here, and therefore only plain concrete was used for the construction of the slab.

Experiment — Flexural specimens SF1-5 (with a span length of 2.254 m or 7.4 ft) were cast with the CFRP SIP form extended halfway into the steel reinforced concrete blocks at both ends in order to simulate the condition of fixity of the deck to the supporting girders (i.e., in the prototype design, this deck system was connected to FRP box girders through dovetail shaped section formed on top of the girder filled with polymer concrete, close to a fixed rigid connection[7]). Shear bond specimens SB1–3 included no end blocks (with a span length of 2.024 m or 6.64 ft) so as to allow for the slippage at the slab-plate interface (see Table 2). All the specimens were simply supported by a roller at one end, a pin at the other, and quasi-statically loaded at mid-span from the top. A double-rod hydraulic actuator was used to apply the load through an elastomeric loading pad up to different service and strength levels in relation to the AASHTO wheel load[13] (HS-20 truck wheel load considering an impact factor of 33% and a load factor of 1.75 for STRENGTH I design level). Flexural specimens SF1–5 behaved quite linear-elastically up to failure (Figure 3). The flexural cracks in SF1 (with a rib spacing of 152 mm or 6 in), SF2 (with a rib spacing of 305 mm or 12 in) and SF3 (with no ribs) first grew vertically near the bottom at mid-span and then propagated diagonally toward the load point due to the combined flexural and shear stresses followed by a sudden diagonal failure crack (e.g., Figure 4) with a similar load capacity of 310 kN (i.e., 69.7 kips due to the restraining effect provided from the concrete end blocks). For the effect of the spacing of the stiffeners, specimen SF4 (with a stiffener spacing of 610 mm or 24 in) and SF5 (no stiffener) had an ultimate capacity about 17% and 43%, respectively, lower than that of the control specimen SF1 (with a spacing of 305 mm or 12 in). The ultimate load level in all 3 specimens exceeded the factored AASHTO wheel load demand (STRENGTH I, as represented by the top dashed line in Figure 3). The compressive strains in concrete and tensile strains in CFRP composites were found to be well within the code and design limit[15,7]. Quasi-static cycles were introduced in the loading protocol of the shear-bond tests on SB1, SB2 and SB3 which had a rib spacing of 152 mm (6 in), 305 mm (12 in), and infinite (no ribs), respectively. Flexural-shear type of crack and horizontal debonding were observed in SB1 and SB2, but SB3 failed in a more flexural manner with the debonding occurring much earlier. The ultimate capacity of SB3 showed 37% lower than that of SB1 and SB2, mainly due to the absence of the interfacial shear ribs. Figure 5 illustrates the load-displacement response for SB1 (where the results for SB2 and SB3 are not shown here in order to allow for a clear comparison with the analytical results to be discussed next).

Analysis — An analytical study was performed using the general-purpose finite element analysis software ABAQUS[16]. The composite deck panel was modeled with 4-node doubly curved general-purpose shell elements with reduced integration points (S4R) and linear elastic orthotropic properties. Eight-node linear brick elements (C3D8) were used for the modeling of the concrete slab. The concrete damaged plasticity model in ABAQUS was used to model the nonlinear behavior of concrete. The sand-bond at the panel interface introduces a friction effect and was modeled using the basic classical Coulomb friction model in ABAQUS combined with the definition of surface interaction. The shear ribs at the interface were modeled with spring elements acting between the node of panel and the node of slab. The behavioral property of the springs was represented by a two-stage elastic bond strength-slippage relationship, where the springs were assumed to behave linear elastically before reaching their ultimate capacity and after that, a sudden failure would occur in them with a sudden load drop and the spring stiffness would go down to zero. The analytically obtained displacement response compared fairly well with the testing data, e.g., Figure 5 illustrates a close correlation for specimen SB1 even though a larger analytical load capacity is found compared to the test. This is mainly due to the fact that multiple cycles were introduced to the loading protocol of the shear bond tests up to failure, which resulted in stiffness degradation in the slab with accumulated damage, and in turn, hindered the specimen from reaching its ultimate capacity (as it should have if monotonic load was applied up to failure). By utilizing the concept of effective crack direction, a graphical visualization of the cracking patterns was obtained as shown in, for example, Figure 6 for SB1, which corresponds closely to the test observations. The direction of the vector normal to the crack plane is parallel to the direction of the maximum principal plastic strain and the length of the colored vector is proportional to the amount of cracking.

Dynamic characteristics

Due to the mass and stiffness difference between FRP composites and conventional steel and concrete materials the dynamic characteristics of bridge deck made from hybrid materials can be different. Dynamic properties, such as natural frequencies and mode shapes, have been found to effectively characterize the state of a structure[17]. A forced vibration testing method, which has been found to serve as a quick and relatively inexpensive method in field application[18], was implemented in this study to characterize the dynamic features of the FRP-concrete deck. Since frequency and mode shape depend on the stiffness of the system, this method was also used to detect the damages undergone in the deck specimen that caused different levels of stiffness degradation.

Test setup — The forced vibration test was conducted on the same shear-bond test specimens (SB1-3) simply supported at the two ends prior to each loading sequence. It was noted that the size of the testing specimens was much smaller than that of the actual full bridge; therefore, the dynamic response of the deck component was expected to be much stiffer with relatively higher magnitude of natural frequency than the full bridge. The setup configuration of the forced vibration test is illustrated in Figure 7, where a PCB drop-weight impact hammer (Modal 086C03) instrumented with an 89 kN (20,000 lb) PCB 200C20 piezoelectric load cell was used for force excitation. Lead ballast was used to increase the impact head weight to 534 N (120 lb). Other major components of the test instrumentation included a 16-channel E Series DAQ Pad device with SCXI-1000 chassis for signal conditioning and SCXI-1520 Strain Gage Module, a Panasonic laptop CF28 to collect time data and conduct further analysis, and a set of uniaxial PCB accelerometers (Model 3701G3FA3G). Fifteen accelerometers were installed to measure the response at the locations as shown in Figure 7. The accelerometers were attached to the deck top and bottom surfaces using aluminum mounting plates. Ten accelerometers were mounted at the bottom surface of the composite deck panel, 9 of which (S1-S9) were lined along the quarter span and mid-span in both longitudinal and transverse directions and the other one (S15) was placed next to the roller support as the reference accelerometer. Five extra accelerometers (S10-S14) were placed on the top surface of the concrete slab along the centerline of the span in alignment with the accelerometers at the bottom surface. Typical accelerations measured by these accelerometers were within the range of ±3g with the sensitivity of 1000mV/g (±5% precision). The frequency range within a precision of ±5% was about 0~100 Hz and 0~150 Hz for the precision of ±10%. The hammer was placed between accelerometers S10 and S11 at the east end (pin support) along the centerline. The hammer location was determined such that it was close enough to the end of the specimen where higher modes of the specimen might also be excited besides the fundamental mode (i.e., placing the hammer closer to the mid-span was likely to excite the first mode only). Following the same loading protocol as the shear-bond tests of SB1-3 (quasi-static cycles), the impact hammer was dropped and the data was acquired after each loading cycle, incurring multiple drops for each of the specimens.

Results — The natural frequency of the first mode (fundamental mode) was obtained as 64.7 Hz, 51.3 Hz and 60.5 Hz for SB1, SB2 and SB3, respectively, before the application of any load to the specimen. It was noticed that this frequency for SB2 was smaller than that for SB1 and SB3 due to the unexpected malfunction in the hammer during the test of SB2. The frequency history is illustrated in Figure 8 at all loading sequences measured by the accelerometers on the top surface of the slab and the bottom surface of the panel. The plot shows a slightly descending trend of the natural frequency before reaching the final failure stage of the specimen, indicating a small degradation of structural stiffness in the specimen as the load increases. After the major failure crack occurred, the frequency of the specimen SB1 dropped to 27 Hz, about a 58% decrease compared to the undamaged stage. The frequency of their second mode was estimated to exceed 150 Hz, which was not detectable due to the limitation in the frequency range of the testing device (a limit of 150 Hz with ±10% precision). The mode shape of the specimen at each load sequence was examined by normalizing the accelerometer measurement with respect to that of the middle accelerometer (S12 for the top ones and S5 for the bottom ones). The typical mode shapes for all load sequences based on the measurements from the top accelerometers are illustrated in Figure 9 for SB1, where the mode shapes in the sequences before the final failure (Sequence 14) are similar. At the end of the failure sequence, the mode amplitude measured from accelerometer S13 was amplified from approximately 0.9 to 1.5 as seen in Figure 9, indicating the occurrence of severe damage at that location. This was also verified by the test observations on the crack pattern as shown on the top of the same figure.

Analysis — The dynamic characteristics of the deck panel were evaluated using the same FEA model constructed in the previous study for static performance. The fundamental frequency (the first mode) was analytically obtained as 68.5 Hz (same for all three cases since the effect from the shear ribs on natural frequency of the system was minor). This is relatively close to the experimentally measured average value of 62.6 Hz based on SB1 and SB3 (Figure 8).

LONG-TERM STRUCTURAL BEHAVIOR

Fatigue response

The strength of highway bridge decks is sensitive to repeated stressing in the material, requiring special attention to fatigue response due to moving traffic loads.

Test setup — The fatigue specimen studied in this research consisted of two continuous spans of 1.22 m (4 ft) wide and 2254 mm (7.4 m) long slabs to include the continuity effect (Figure 10a). A set of CFRP mesh layers was placed near the top surface of the slab over the middle one-third region, providing the tensile reinforcement for that negative bending moment area. The specimen was simply supported and loaded in a sinusoidal waveform by two patch loads of 84 kN (18.9 kips) placed 1828.8 mm (6 ft) apart via two double-rod hydraulic actuators to simulate one axle of the AASHTO truck wheel load. The specimen experienced 2.1 million cycles of fatigue service load and 250,000 cycles of doubled fatigue service load followed by 10,000 cycles of tripled fatigue load (Figure 10b). The specimen was then monotonically loaded up to failure.

Results — Hairline cracks were found on the top surface of the slab above the middle support at the end of the 2 million cycles of fatigue service load (negative bending moment region where layers of tensile fiber mesh were embedded). The crack width was fairly small within the serviceability limit state per code requirement. No tensile cracks were observed on the vertical sides of the specimen. The maximum deflection of the structure under fatigue service load was found to be within the deflection-to-span ratio limit (L/800), satisfying the serviceability limit state with respect to the deflection[12]. The structure was found to suffer no stiffness degradation during the first 2 million cycles of fatigue service load based on the small variation in the observed structural response. However, a substantial degradation of 37.6% was found during the subsequent 250,000 cycles of doubled fatigue service load (i.e., 336 kN instead of 168 kN, or 75.5 kips instead of 37.8 kips) and 44% during the further 10,000 cycles of tripled load (i.e., 504 kN instead of 168 kN, or 113.3 kips instead of 37.8 kips), as seen in Figure 11, indicating the higher the magnitude of the wheel load, the larger the amount of degradation in the system. The residual displacement in the system under all the fatigue load conditions was found to be insignificant and displayed a largely elastic and stable manner (Figure 11), indicating no slippage at the slab-deck interface. The tensile strain and compressive strain experienced in the FRP composites and concrete material were well below the design allowables[19]. The carbon fiber mesh that was embedded in the middle support was found to be effective in providing the tensile strength for concrete in the continuity region (negative moment region). More discussions on these fatigue test results are available in a companion paper[19].

Temperature effect

Since each material has its own thermal expansion coefficient, structural members consisting of different materials experience different stress and strain distribution and deformation that are introduced due to temperature variations. To maintain the internal force equilibrium and compatibility in the deck member, this temperature change causes shear stress at the interface between the concrete and the CFRP panel. The magnitude of this shear stress was evaluated in this study in order to ensure an acceptable bond level at the interface without any failure. Since the laminate of the bottom reinforcement was primarily made of 8 layers of unidirectional carbon/epoxy FRP (same orientation and material), the residual interlaminar stresses between the adjacent CFRP layers caused by temperature change within the plate were negligible and therefore not considered herein.

Assumptions — The previously validated FEA model on half-span deck system with end restraint from girders was used for the study of temperature effect. ABAQUS software was utilized for the simulation. The thermal expansion characteristics of concrete material were affected mainly by the types and proportions of aggregates used and the degree of saturation of the concrete. The thermal expansion coefficient of normal density concrete varies within $5.4–14.4\times10^{-6}$/ºC (i.e., $3–8\times10^{-6}$/ºF, AASHTO[13], ACI 209[20]) assuming an isotropic property. A typical value of 10.8×10^{-6}/ºC (6×10^{-6}/ºF) was used in the current study. The CFRP composites, on the other hand, exhibit an anisotropic property. The longitudinal and transverse coefficient of thermal expansion of the bottom plate was estimated as 0.995×10^{-6}/ºC (0.553×10^{-6}/ºF) and 22.9×10^{-6}/ºC (12.7×10^{-6}/ºF), respectively, based on the laminate design. Other basic assumptions adopted here included: 1) the FRP composite material was linear elastic and concrete behaves nonlinearly; and 2) the modulus of elasticity of concrete was estimated by $4800\sqrt{f_c'}$ (where f_c' was the compressive strength of concrete in the unit of MPa[12]). Two types of temperature variation were considered in the study: a) a uniform temperature where the temperature range was taken as the difference between the extended lower (–12ºC or 10.4ºF for concrete) or upper boundary (27ºC or 80.6ºF for concrete) and the assumed base construction temperature (20ºC or 68ºF) under moderate climate[12] (resulting in a maximum temperature change of ±32ºC or –25.6~89.6ºF for the analysis); and b) a temperature gradient where the vertical temperature gradient in the deck was determined from a bi-linear relation as specified in AASHTO[13]. The positive temperature values were taken for Zone 1 (Figure 12) and the negative values were obtained by multiplying the positive values by –0.3 for plain concrete decks.

Results — The interfacial shear stress in both the longitudinal and transverse directions due to temperature variation was examined, where the shear stress at the interface in each direction was determined from the normal stress difference between the composite deck panel and the concrete slab at the same location. It was found that the maximum shear stresses under uniform temperature change and temperature gradient per AASHTO were in the longitudinal direction, as seen in Table 3 with the maximum level of 15.5 MPa (2.25 ksi) under a uniform temperature increase of 32ºC (89.6ºF). These stress levels are smaller than the typical shear strength of epoxy resin[21] in the shear ribs, i.e., 34 MPa (4.93 ksi), implying that shearing-off failure in the interfacial ribs is unlikely to occur under the code design uniform and gradient temperature variation. Figure 12 displays the normal stress contour in both concrete slab and FRP panel along the longitudinal direction under a uniform temperature increase of 32ºC (89.6ºF) and the previously defined positive temperature gradient. The tensile stress in concrete was found to be significantly lower than concrete cracking strength. Further parametric study also showed that under a uniform temperature change between –50ºC and 50ºC (i.e., –58ºF~122ºF), the maximum shear stress at the concrete-CFRP interface increased fairly linearly as the temperature gradually increased, and the vice versa (the results of which were not graphically presented here). Under this high temperature variation, the interfacial shear stress level was found to be within the shear strength limit of the ribs. It was also observed from the deformed shape that a temperature increase caused an upward convex shape (in the longitudinal direction) while a temperature decrease introduced a downward concave shape, implying more expansion or contraction experienced in the concrete slab than that in the CFRP plate. This can be explained by the fact that the longitudinal thermal expansion coefficient in the concrete (10.8×10^{-6}/ºC) is almost 10 times larger than that of the CFRP plate (0.995×10^{-6}/ºC), and this difference is much less critical in the transverse direction.

Shrinkage and creep effect

It is known that the commonly used fiber and matrix (e.g., carbon and epoxy) have low shrinkage characteristics Any small volume change due to this low shrinkage takes place at and shortly after the fabrication process. This will not likely affect the stress distribution over the FRP-concrete deck system (especially the CFRP panels are pre-fabricated long before being shipped to the construction site). Therefore, concrete remains to be the only material that introduces shrinkage stresses into the system. However, this shrinkage effect is significantly alleviated by mixing the concrete with fibrillated polypropylene fibers as proposed in the design. The hybrid FRP-concrete deck system was thus assumed to be shrinkage free. For the creep effect, materials with higher elastic modulus generally show smaller creep strains since the elastic modulus depends on the atomic bond[22]. The carbon fiber has an elastic modulus more than 50 times larger than that of the epoxy[21] and hence their creep effect can be ignored. Moreover, the bottom plate of the CFRP panel is made primarily from unidirectional carbon fiber, which results in a high fiber volume fraction with negligible contribution from the matrix. Therefore, no time-dependent strain increase due to CFRP creep needed to be considered. The time-dependent increase of strain in concrete due to creep can be estimated using standard creep models[20], which were not implemented in the current study.

CONCLUSIONS

The utilization of FRP composites for structurally integrated stay-in-place formwork for concrete bridge decks can not only improve the durability of the system, but also simplify and accelerate bridge construction process. This paper presents rather extensive experimental characterization and development of appropriate analytical methods of a new hybrid deck system that uses stiffened CFRP deck panel as both permanent formwork and primary tensile reinforcement for concrete slab. Research results showed that the system exhibited fairly good ultimate flexural capacity with typical concrete failure instead of catastrophic FRP brittle failure. The spacing of the interfacial stiffeners and shear ribs at the CFRP surface was sufficient in transferring the shear from the slab to the reinforcing plate. Fatigue was experimentally found to be a non-governing limit state for the design. The fundamental natural frequency of the system was characterized by using a forced vibration testing method, which also detected the major failures in the slab. The long-term behavior due to environmental effects such as temperature, shrinkage and creep were found to be satisfactory. Design charts and simplified equations that were developed for practical use are available in a separate paper[10].

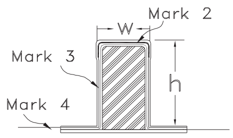
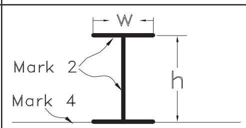
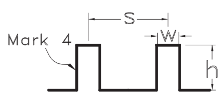
Although promising, several areas are needing future work to promote the use of this structurally integrated stay-in-place FRP form in bridge deck construction. Those include: a) experimental study on the temperature effect; b) investigation of practical bonding alternatives that provide sufficient interface bond between FRP and concrete; c) durability of the system under extreme environmental conditions; d) ductility improvement in the system achieving reasonable seismic performance; e) connections of the deck system to its supporting structural members (e.g., girders) that allows easy replacement in the future; and f) more demonstration projects illustrating the feasibility of the system in the field.

REFERENCES

1. Brailsford, B., Mikovich, S.M., and Hopwood, T., “Definition of Infrastructure Specific Markets for Composite Materials,” *Technical Rep., Project P93-121/A573*, Northwestern University, Evanston, Illinois, 1995.
2. Bettigole, N.H., “Replacing Bridge Decks,” *Civil Engineering*, V. 60, No. 9, 1990, pp. 76-77.
3. Hassan, T., Abdelrahman, A., Tadros, G., and Rizkalla, S., “Fibre Reinforced Polymer Reinforcing Bars for Bridge Decks,” *Can. J. Civ. Eng.*, V. 27, 2000, pp. 839-849.
4. ACI, Guide for the Design and Construction of Concrete Reinforced with FRP Bars. *ACI 440.1R-03*, American Concrete Institute, 2004.
5. Yost, J.R., and Schmeckpeper, E.R., “Strength and Serviceability of FRP Grid Reinforced Bridge Deck,” *Journal of Bridge Engineering*, V. 6, No. 6, 2001, pp. 605-612.
6. Ishizaki, S., Kubo, K., and Matsui, S., “An Experimental Study on Durability of FRP-RC Composite Deck Slabs of Highway Bridges,” *Proceedings of the 3rd International Conference on Concrete Under Severe Conditions: Environment & Loading*, Vancouver, Canada, 2001, pp. 933-940.
7. Cheng, L., Zhao, L., Karbhari, V.M., Hegemier, G.A., and Seible, F., “Assessment of a Steel-Free Fiber Reinforced Polymer-Composite Modular Bridge System,” *Journal of Structural Engineering*, ASCE, V. 131, No. 3, 2005, pp. 498-506.
8. Berg, A.C., Bank, L.C., Oliva, M.G., and Russell, J.S., “Construction of a FRP Reinforced Bridge Deck on US Highway 151 in Wisconsin,” *Proceeding of the 83rd Annual Transportation Research Board Meeting*, CD-ROM, Washington, DC, V. 25, Jan 11-14, 2005.
9. Cheng, L., and Karbhari, V.M., “New Bridge Systems using FRP Composites and Concrete: A State-of-the-Art Review,” *Journal of Progress in Structural Engineering and Materials*, V. 8, 2006, pp. 143-154.
10. Cheng, L., and Karbhari, V.M., “Design Approach for a FRP Structural Formwork Based Steel-Free Modular Bridge System,” *International Journal of Structural Engineering and Mechanics*, Techno-Press, V. 24, No. 5, 2006, pp. 561-584.
11. Naaman, A.E., Shah, S.P., and Thorne, J.L., “Some Developments in Polypropylene Fibres for Concrete,” *Fibre Reinforced Concrete*, ACI SP-81, American Concrete Institute, Detroit, 1984, pp. 386-396.
12. ASTM, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, PA, 2000.
13. *AASHTO LRFD Bridge Design Specifications*, 3rd Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 2004.
14. CHBDC, *Canadian Highway Bridge Design Code – Section 16*, Fibre-Reinforced Structures, final draft, 1996.
15. ACI 318-95, Building Code Requirements for Reinforced Concrete (ACI 318-95) and Commentary (ACI 318R-95), *American Concrete Institute*, Detroit, 1995.
16. ABAQUS/Standard User’s Manual, Version 6.5, ABAQUS Inc., 2007.
17. Ewins, D.J., *Modal Testing: Theory and Practice*, New York: Wiley, 1984.
18. Stubbs, N., Choi, S., Bolton, R.W., and Park, S., “Nondestructive Evaluation of District 08 Bridges using Modal Parameters,” Final Report, FHWA/CA/SD-2001/07, Texas Engineering Experiment Station, College Station, Texas, 2001.
19. Cheng, L., and Karbhari, V.M., “Fatigue Behavior of a Steel-Free FRP-Concrete Modular Bridge Deck System,” *Journal of Bridge Engineering*, ASCE, V. 11, No. 4, 2006, pp. 474-488.
20. ACI Committee 209, “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures.” *American Concrete Institute*, 1992.
21. Kaw, A.K., *Mechanics of Composite Materials*, CRC Press LLC, 2nd edition, 2006.

22. Deskovic, N., "Innovative Design of FRP Composite Members Combined with Concrete," PhD thesis, Department of Civil and Environmental Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1993.

Table 1 — Design alternatives for CFRP deck panel (constant spacing of stiffeners = 305 mm or 12 in)

Design Scheme	Cross-Sectional Shape	Dimension of Stiffener *w*×*h*, mm [in]	Thickness of CFRP, mm [in]	CFRP Amount*	Flexural Stiffness for a 610 mm (2 ft) wide Section, kN-m² [kip-in²]
Rectangular (prototype)		41×105 [1.614×4.134]	Top (Mark 2): 2.4 [0.094] Side (Mark 3): 1.75 [0.069] Bottom (Mark 4): 6.3 [0.248]	100%	194.5 [67747.2]
I-beam		62×105 [2.441×4.134]	Top/side (Mark 2): 2.4 [0.094] Bottom (Mark 4): 6.3 [0.248]	97%	195.9 [68234.9]
Corrugated		41×74 [1.614×2.913]	Uniform (Mark 4): 6.3 [0.248]	106%	199.7 [69558.4]

* Normalized by the original design of rectangular stiffeners.

Table 2 — Matrix of test program

Specimen	Size: length×width×thickness, m [in]	Spacing of Rectangular Stiffeners, mm [in] (total # of stiffeners)	Spacing of Shear Ribs, mm [in]	End Condition
SF1*†	2254×610×203 [88.7×24×8] (CFRP plate only)	305 [12] (2)	152 [6]	Constructed with steel reinforced concrete blocks at both ends
SF2		305 [12] (2)	305 [12]	
SF3		305 [12] (2)	∞ (no ribs)	
SF4		610 [24] (1)	152 [6]	
SF5		∞ (0)	152 [6]	
SB1*†	2024×610×203 [79.7×24×8] (CFRP plate only)	305 [12] (2)	152 [6]	No end blocks constructed
SB2			305 [12]	
SB3			∞ (no ribs)	
FT1†	(2@2254)×1220×203 [2@88.7×48×8] (CFRP plate + fiber mesh)	305 [12] (4)	152 [6]	2 end blocks + 1 middle block

* Control panel as originally designed;
† SF – Flexural specimen; SB – Shear bond specimen also used in the forced-vibration test; FT – Fatigue specimen.

Table 3 — Maximum interfacial shear stress due to temperature variation

Maximum Interfacial Shear Stress	Uniform Temperature		Temperature Gradient	
	ΔT = 32ºC [89.6ºF]	ΔT = –32ºC [–25.6ºF]	Positive	Negative
Longitudinal	15.5 MPa [2.25 ksi]	15.4 MPa [2.24 ksi]	1.2 MPa [0.17 ksi]	0.35 MPa [0.05 ksi]
Transverse	2.7 MPa [0.39 ksi]	2.7 MPa [0.39 ksi]	0.6 MPa [0.09 ksi]	0.17 MPa [0.02 ksi]

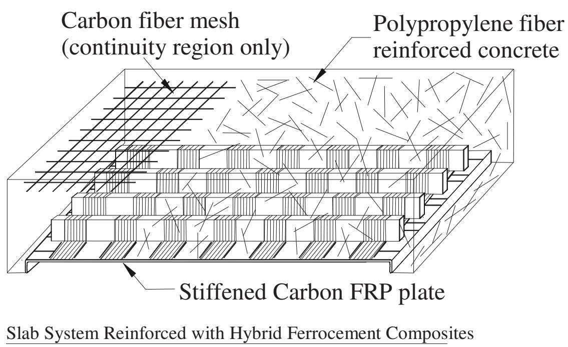


Figure 1 — Design concept of hybrid FRP-concrete bridge deck system

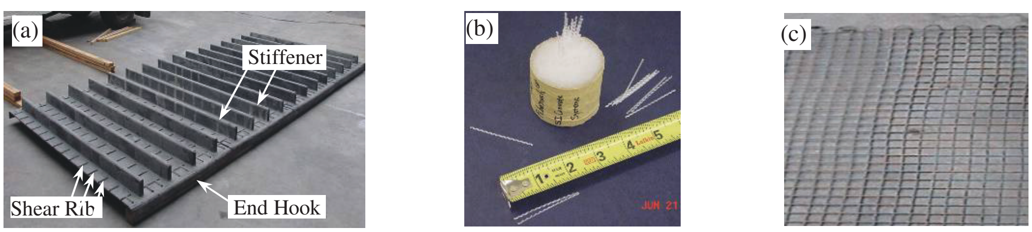


Figure 2 — Material details: (a) CFRP plate; (b) polypropylene fiber; (c) carbon fiber mesh

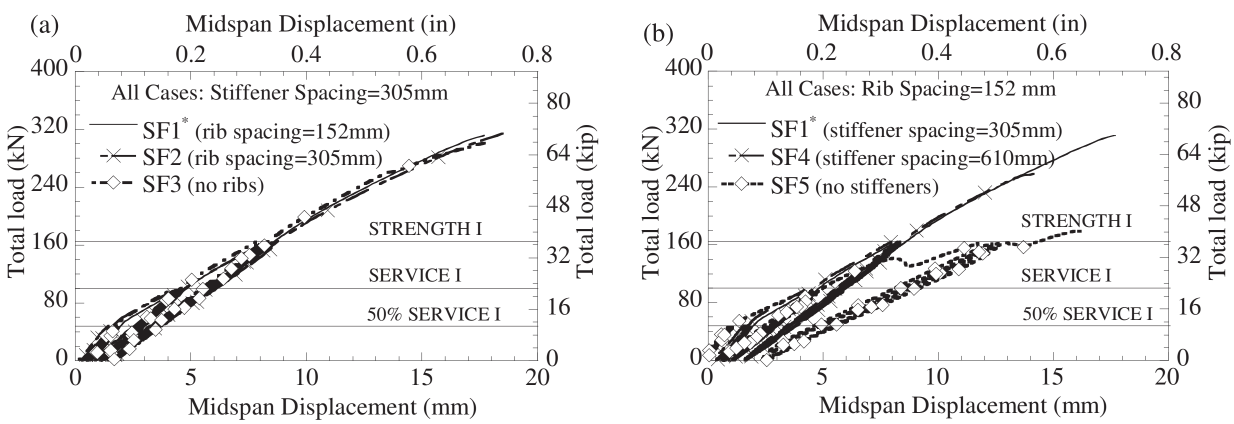


Figure 3 — Load versus midspan displacement for static flexure tests on: (a) SF1, SF2 and SF3; (b) SF1, SF4 and SF5