

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

2 Cheng and Karbhari

Lijuan “Dawn” Cheng is an Assistant Professor in the Department of Civil and Environmental Engineering at the University of California, Davis. She received her PhD from the University of California, San Diego. She is a member of ACI Committee 440, Fiber Reinforced Polymer Reinforcement, ASCE, SAMPE, and IIFC. Her current research interests include strengthening of masonry structures using fiber reinforced polymer composites, near surface mounted technique, large-scale testing and numerical simulations.

Vistasp M. Karbhari is a Professor in the Department of Structural Engineering at the University of California, San Diego. He is Chair of the technical advisory committee of American Composites Manufacturers Association, Vice-Chair of the advanced materials research committee of ASCE’s construction institute’s materials directorate, and a member of ACI Committee 440. His research interests include infrastructure rehabilitation and application of composites to infrastructure renewal, structural health monitoring, durability and manufacturing of composites.

INTRODUCTION

Concrete bridge decks have been the most deficient structure among all bridge components primarily due to the corrosion associated with their steel reinforcement¹. Statistics showed that U.S. bridges last 68 years on average but their decks last 35 years, only about half of their superstructure². 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 tendons³⁻⁴, 2-dimensional and 3-dimensional gratings and grids⁵. FRP continuous plates have also been used for durability considerations⁶ and more recently for both tensile reinforcement⁷⁻⁸ 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-art⁹: 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 elsewhere¹⁰; 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 hydrophobic¹¹, 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 girders⁹ 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 $[\text{C}/\text{E}/\text{C}_2/\text{E}/\text{C}]_s$ (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, $41 \times 105 \text{ mm}$) and the corrugated design (with 2 corrugations of the size of $41 \times 74 \text{ mm}$ with the same spacing) can possibly be used to replace the rectangular design ($41 \times 105 \text{ mm}$). 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 D3039-76¹² 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

4 Cheng and Karbhari

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 AASHTO¹³.

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, CHBDC¹⁴). 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 connection⁷). 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 load¹³ (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 ABAQUS¹⁶. 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 structure¹⁷. A forced vibration testing method, which has been found to serve as a quick and relatively inexpensive method in field application¹⁸, 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 $\pm 3g$ with the sensitivity of 1000mV/g ($\pm 5\%$ precision). The frequency range within a precision of $\pm 5\%$ was about 0~100 Hz and 0~150 Hz for the precision of $\pm 10\%$. 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 $\pm 10\%$ 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,

6 Cheng and Karbhari

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 deflection¹². 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 allowables¹⁹. 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 paper¹⁹.

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 $5.4\text{--}14.4 \times 10^{-6}/^\circ\text{C}$ (i.e., $3\text{--}8 \times 10^{-6}/^\circ\text{F}$, AASHTO¹³, ACI 209²⁰) assuming an isotropic property. A typical value of $10.8 \times 10^{-6}/^\circ\text{C}$ ($6 \times 10^{-6}/^\circ\text{F}$) 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 $0.995 \times 10^{-6}/^{\circ}\text{C}$ ($0.553 \times 10^{-6}/^{\circ}\text{F}$) and $22.9 \times 10^{-6}/^{\circ}\text{C}$ ($12.7 \times 10^{-6}/^{\circ}\text{F}$), 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 (-12°C or 10.4°F for concrete) or upper boundary (27°C or 80.6°F for concrete) and the assumed base construction temperature (20°C or 68°F) under moderate climate¹² (resulting in a maximum temperature change of $\pm 32^{\circ}\text{C}$ or $-25.6 \sim 89.6^{\circ}\text{F}$ 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 AASHTO¹³. 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 32°C (89.6°F). These stress levels are smaller than the typical shear strength of epoxy resin²¹ 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 32°C (89.6°F) 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 -50°C and 50°C (i.e., $-58^{\circ}\text{F} \sim 122^{\circ}\text{F}$), 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 ($10.8 \times 10^{-6}/^{\circ}\text{C}$) is almost 10 times larger than that of the CFRP plate ($0.995 \times 10^{-6}/^{\circ}\text{C}$), 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 bond²². The carbon fiber has an elastic modulus more than 50 times larger than that of the epoxy²¹ 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 models²⁰, 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

8 Cheng and Karbhari

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 paper¹⁰.

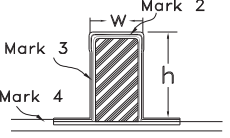
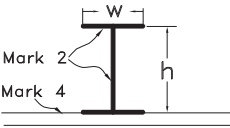
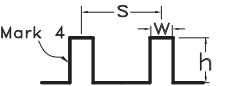
Although promising, several areas are needing future work to promote the use of this structurally integrated stay-in-place FRP form in bridge deck construction. Those include: a) experimental study on the temperature effect; b) investigation of practical bonding alternatives that provide sufficient interface 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 $w \times h$, mm [in]	Thickness of CFRP, mm [in]	CFRP Amount*	Flexural Stiffness for a 610 mm (2 ft) wide Section, kN-m^2 [kip-in^2]
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

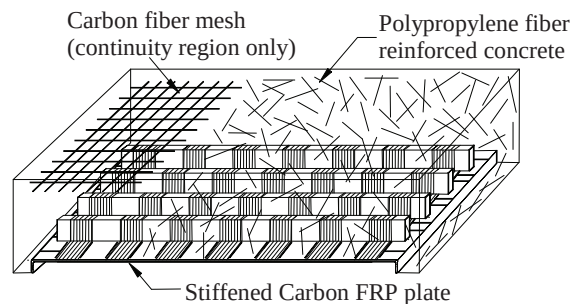
Specimen	Size: length×width×thickness, m [in]	Spacing of Rectangular Stiffeners, mm [in] (total # of stiffeners)	Spacing of Shear Ribs, mm [in]	End Condition
SF1 ^{*†}	2254×610×203 [88.7×24×8] (CFRP plate only)	305 [12] (2)	152 [6]	Constructed with steel reinforced concrete blocks at both ends
SF2		305 [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	
	$\Delta T = 32^{\circ}\text{C}$ [89.6 $^{\circ}\text{F}$]	$\Delta T = -32^{\circ}\text{C}$ [-25.6 $^{\circ}\text{F}$]	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]



Slab System Reinforced with Hybrid Ferrocement Composites

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

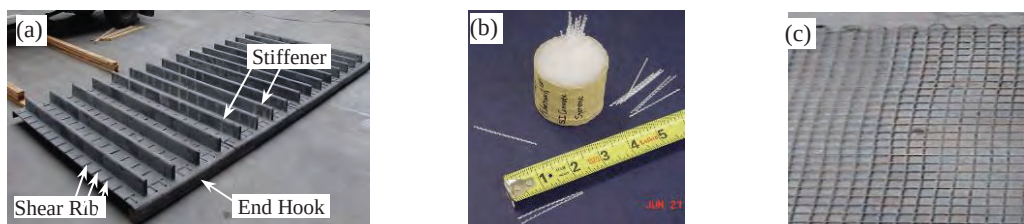


Figure 2 — Material details: (a) CFRP plate; (b) polypropylene fiber; (c) carbon fiber mesh

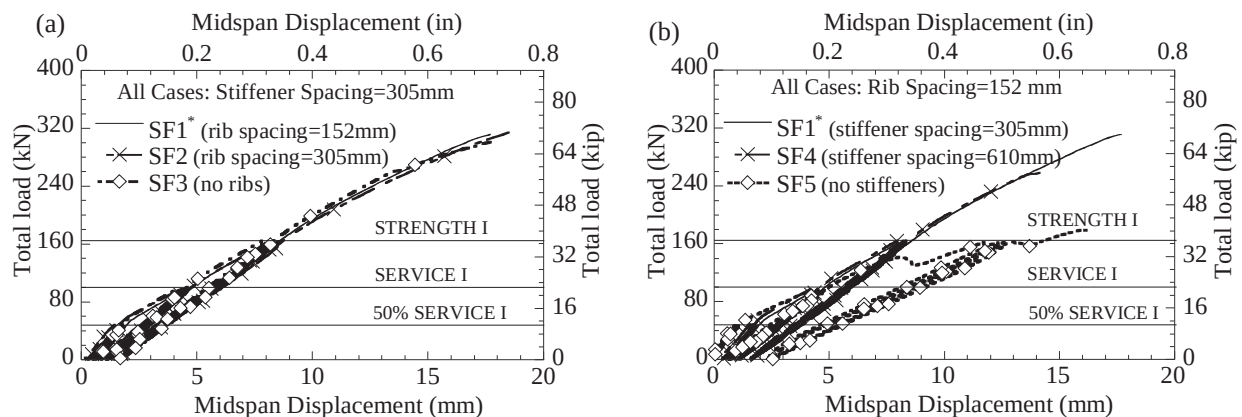


Figure 3 — Load versus midspan displacement for static flexure tests on: (a) SF1, SF2 and SF3; (b) SF1, SF4 and SF5

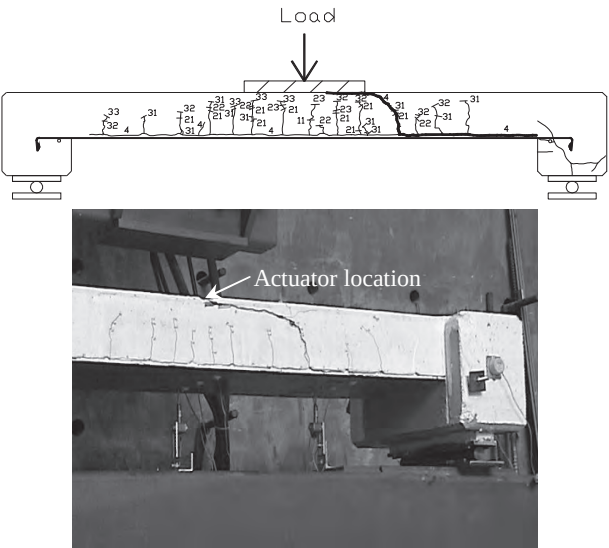


Figure 4 — Typical failure mode in static flexure test

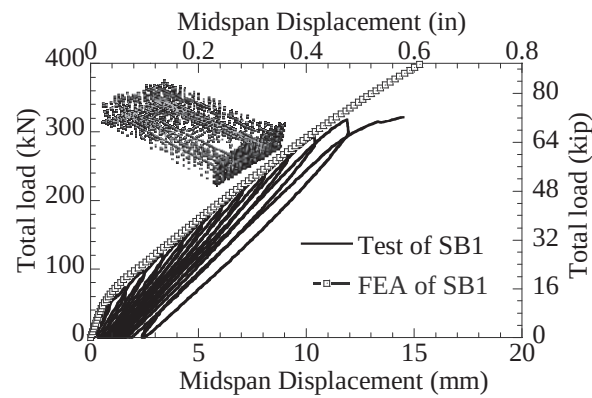


Figure 5 — Load versus midspan displacement for shear-bond test SB1
Comparison of load-displament response

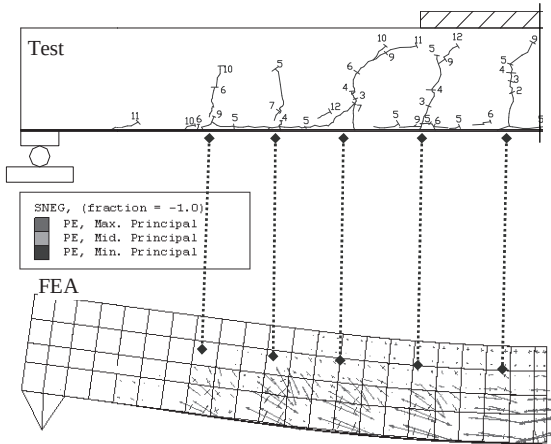


Figure 6 — FEA verification of shear-bond test (SB1)

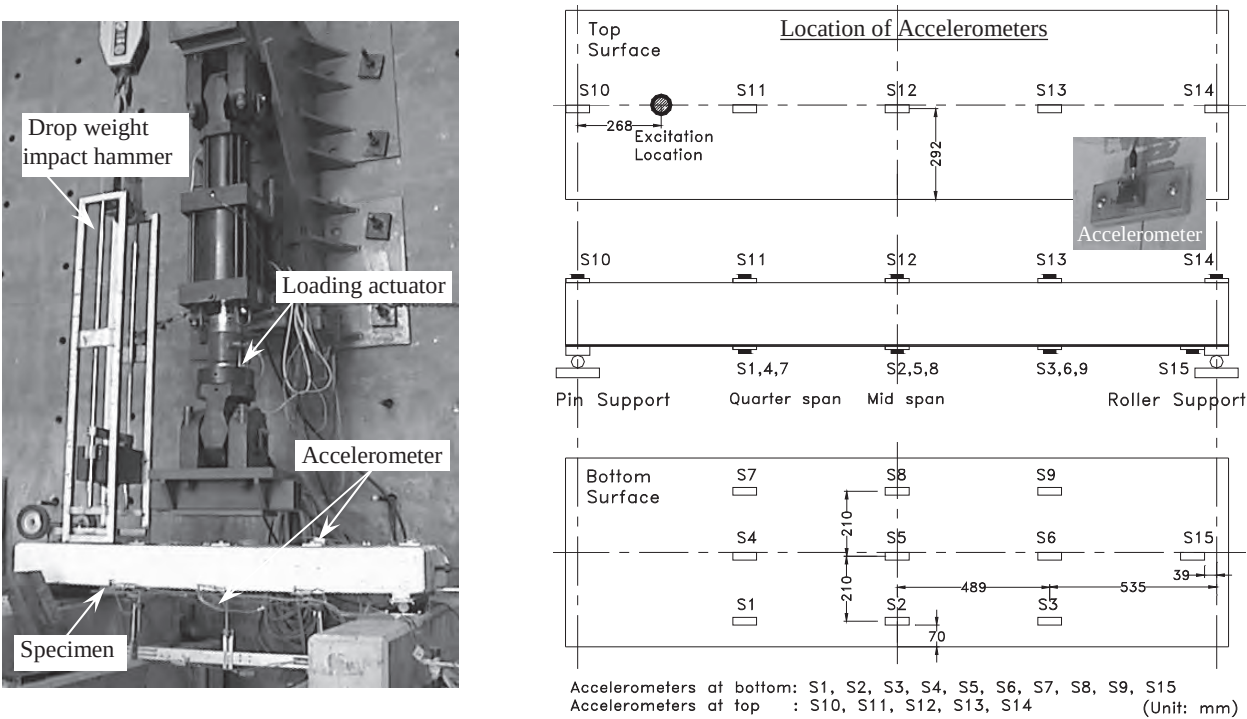


Figure 7 — Setup of the forced vibration test for dynamic characterization

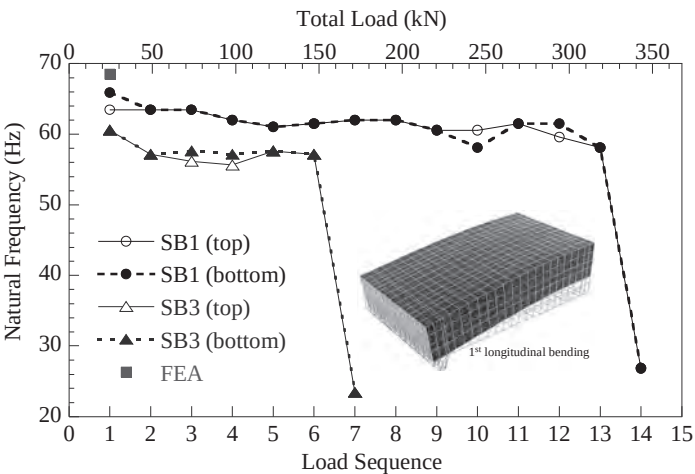


Figure 8 — Fundamental frequency history

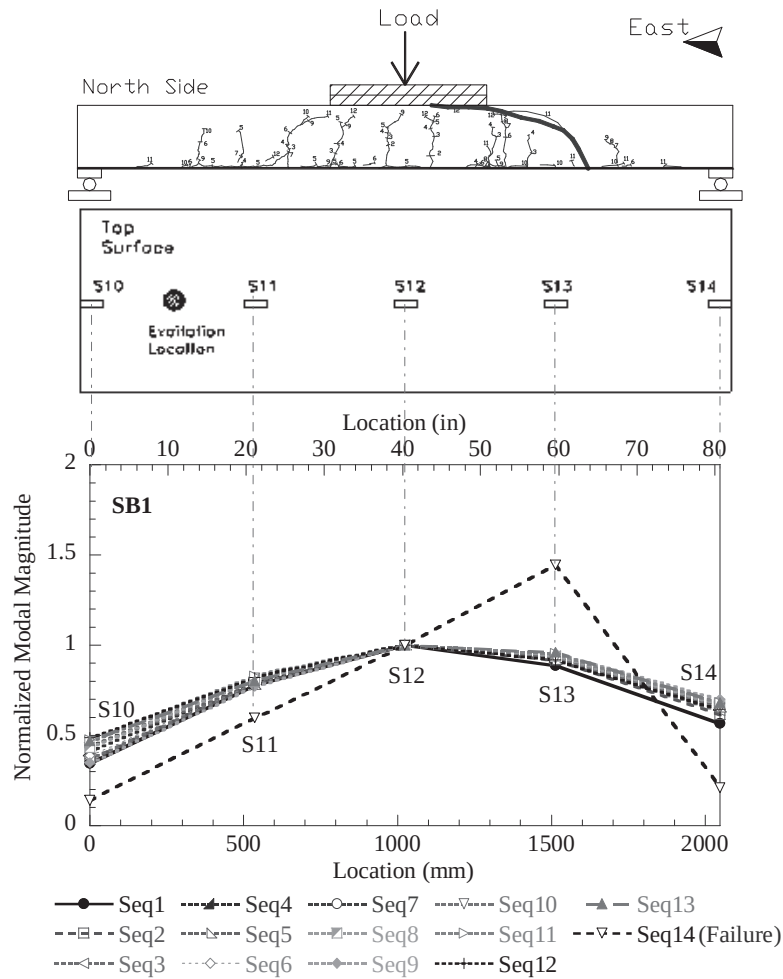


Figure 9 — Mode shape measured from accelerometers on the top

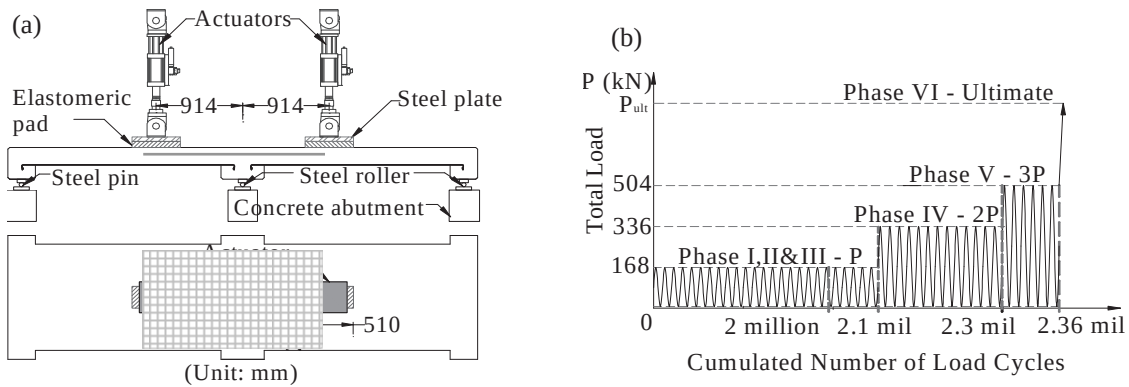


Figure 10 — Fatigue test: (a) setup configuration; and (b) loading protocol

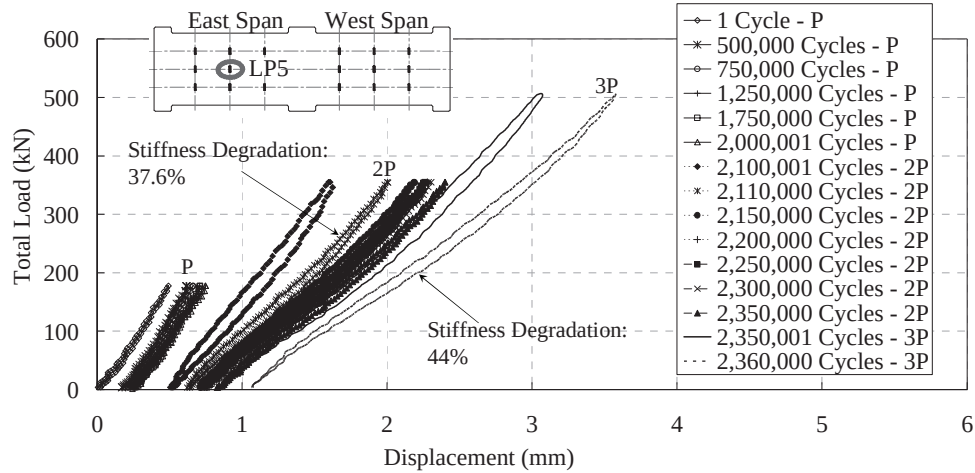


Figure 11 — Load versus midspan displacement response for fatigue test

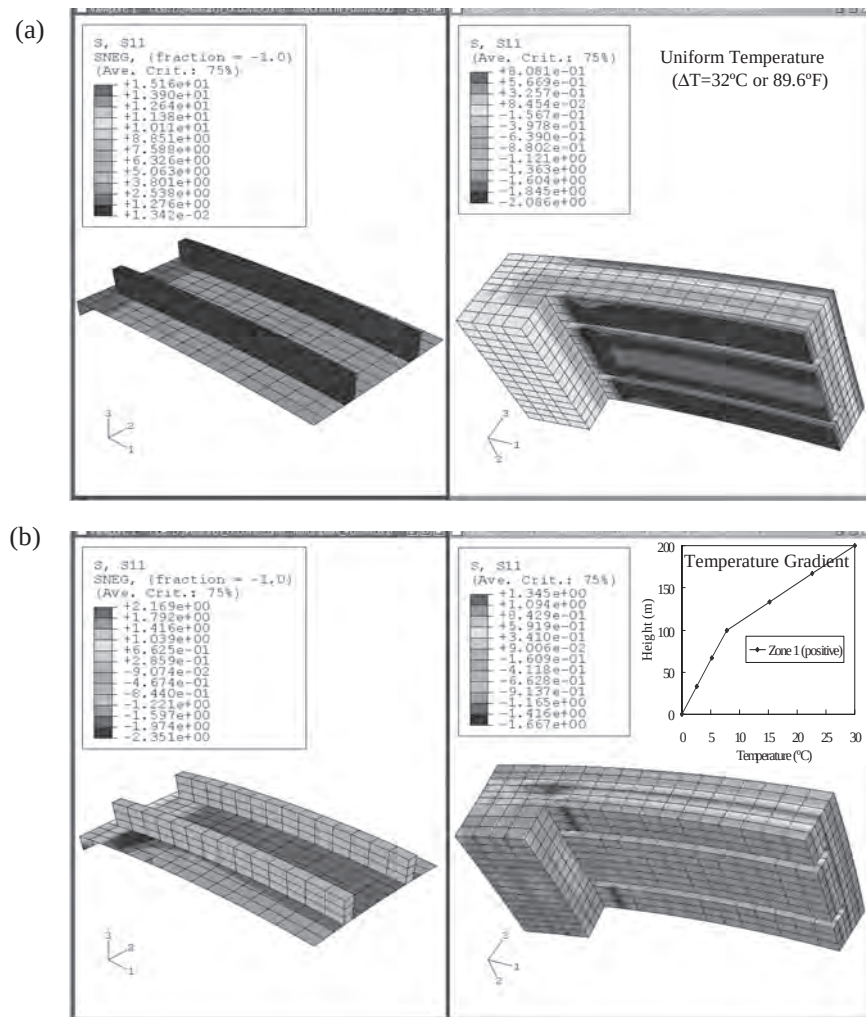


Figure 12 — Longitudinal stress contours due to: (a) uniform $\Delta T = +32^\circ\text{C}$, (b) positive temperature gradient

Flexural Behavior of Hybrid Composite Girders for Bridge Construction

by A. Elmahdy, R. El-Hacha, and N. Shrive

Synopsis: A continuing goal for the civil engineering industry is the design of lightweight flexural elements with high capacity and high resistance to corrosion. The behavior of hybrid composite hollow-box beams was investigated experimentally and analytically to determine the feasibility and effectiveness of such structural sections as the main flexural members in bridge construction. The section was composed of a glass fiber-reinforced polymer (GFRP) pultruded hollow section, reinforced with one layer of steel-reinforced polymer (SRP) sheet or two layers of carbon fiber-reinforced polymer (CFRP) sheet bonded to the bottom flange to carry the tensile stresses. A 54 mm (2 1/8 in) thin layer of ultra-high-performance concrete (UHPC) was cast on top of the section to carry the compressive stresses. In addition to the structural contribution of the GFRP, the box section will act as a permanent stay-in-place form for the concrete. All these materials have superior strength and durability compared to conventional construction materials, so they can be used in much smaller dimensions than traditionally. The materials in the section were assembled using different types of epoxy. Ease of construction and minimum production cost were included in the aims for the system being developed.

Three beams have been tested to date: two beams had one layer of externally bonded SRP sheet on the tension side with spans of 6000 mm (236 in.) and 5100 mm (200 in.) while the other beam had two layers of CFRP sheets with a span length of 6000 mm (236 in.). Simplified structural analysis of the beams has been implemented. The results of the analyses of the three hybrid beams were compared to the experimental tests, and good agreement between the analytical and experimental work was achieved. Adding UHPC and SRP increased the GFRP section capacity significantly. The preparation and assembly stages of the hybrid sections are presented and the flexural behavior of the beams under monotonic static loading are compared at various stages of loading.

Keywords: carbon; composite; epoxy; externally bonded; glass; hollow section; hybrid; steel; ultra-high strength

Biographical sketch for authors

Ayman Elmahdy is currently a Ph.D. candidate in the department of civil engineering, University of Calgary, Calgary, AB, Canada. He has been awarded B.Sc. (2000) and M.Sc. (2004) degrees in structural engineering from Ain Shams University, Cairo, Egypt, where he worked as a lecturer assistant for 4 years. Elmahdy participated in the design of several concrete and steel structures in Egypt, and his research interests are in bridges and applications of new materials in structures.

ACI Member **Raafat El-Hacha** is an Assistant Professor and Alberta Ingenuity New Faculty in the Department of Civil Engineering at the University of Calgary, Canada. He obtained his PhD degree in 2001 from Queen's University, Canada. He is member of ACI Committee 440, Fiber Reinforced Polymer (FRP) Reinforcement, and chair of Sub-Committee 440-I, FRP Prestressed Concrete. He is an associate member of joint ACI-ASCE 423, Prestressed Concrete, ACI 408, Bond and Development of Reinforcement, and ACI 437, Strength Evaluation of Concrete Structures. His research interests include strengthening concrete and steel structures and reinforcing/prestressing concrete structures with FRP material, blast mitigation and repair using FRP, use of high-strength materials in new construction. He is member of the CSCE, ASCE, PCI, PTI, IIFC, and project leader in ISIS Canada Research Network.

Nigel Shrive is the Killam Memorial Professor, Department of Civil Engineering, University of Calgary, Calgary, AB, Canada. He obtained his bachelor and doctoral degrees from Oxford University. He has wide-ranging research interests in civil engineering and biological materials and structures. He serves on the CSA committees for the structural design of masonry, and masonry construction while chairing the mortar and grout standard committee. He is fellow the Institution of Civil Engineers and also of the CSCE. Nigel is a project leader in the Intelligent Sensing for Innovation Structures (ISIS) Canada Research Network.

INTRODUCTION

Developments in civil engineering construction worldwide have created the need for new construction materials to support structural progress, especially in case of severe construction requirements like large span girders or high-rise buildings. The demonstrated advantages of new materials such as Fiber and Steel Reinforced Polymers, and Ultra High Performance Concrete, have shown that these materials will play an important role in future civil engineering projects. The outstanding engineering characteristics of these materials, mainly their high strength-to-weight ratios and high corrosion resistance, can change much in the construction and rehabilitation of infrastructure. However, the use of new materials in structures requires appropriate design philosophies, guidelines and detailing to be established. Also the design methodology should ensure economic use of the materials. This prognosis for the application of advanced composite materials in civil engineering construction indicates that critical technical and implementation issues need to be resolved prior to broad acceptance of these materials by the civil engineering design and construction community.

The work described here is concerned with developing and understanding of the behavior of an innovative beam composed of Ultra High Performance Concrete (UHPC) in the compression zone with Steel Reinforced Polymer (SRP) or Carbon Fibre Reinforced Polymer (CFRP) sheets to resist the tension. These superior materials are connected via a Glass Fiber Reinforced Polymer (GFRP) rectangular box beam. This box beam is also responsible for carrying the shear stresses in the beam. This combination of materials is expected to result in low-weight, low-maintenance, high performance elements at acceptable costs. The presence of UHPC on the top side of the beam supports the top GFRP flange which will avoid the buckling of this flange under high compressive stresses. The high deflections that would occur if the GFRP box section was used alone are greatly reduced by the addition of the high stiffness materials.

RESEARCH SIGNIFICANCE

The combination of the UHPC with SRP or CFRP and GFRP materials in the proposed innovative beam is expected to result in low-weight, low-maintenance, high performance elements at acceptable costs. The ability to optimize the structure based on the material properties of each constituent material is reflected in the cost-effectiveness of hybrid structural systems. It is expected that applying these new materials effectively will lead to faster construction and

less traffic interruption for bridges due to their lightweight. Lower life-cycle costs are also expected due to reduced maintenance. Good aesthetics should occur because of the small dimensions. The objectives are to achieve technical efficiency with competitive economical results, and to help understanding the influence of using new materials in bridges and the development of new lightweight, high strength structures.

LITERATURE REVIEW

The behavior of composite sections where FRPs have been combined with conventional materials to reduce the initial costs has been studied widely in recent years. For example, Aref et al., (2005) conducted experimental and finite element analysis on a hybrid FRP-concrete structural system, for which the authors also derived simplified equations. The bridge was a simply supported, 18.3 m (60.4 ft) single span, one-lane structure. The cross section consisted of three trapezoidal GFRP box beams with a concrete layer inside the top flange as shown in [Figure 1](#). A similar concept was first proposed and investigated by Deskovic et al. (1995b), who studied a simple beam comprised of a GFRP box section with a layer of concrete on top and a layer of CFRP on the bottom. The authors performed experimental tests on large-scale specimens and found good agreement with finite element models. A preliminary design procedure was proposed for the hybrid section ([Figure 2](#)). The design proposal was based on stiffness, strength and ductility requirements. The researchers then examined the long-term behavior of the hybrid section (Deskovic et al. 1995a). The time-dependent behaviors like shrinkage, creep and fatigue were studied through consideration of compatibility and equilibrium of the section after redistribution of the strains and stresses. In a third variation of this theme, the behavior of GFRP tubes acting compositely with a high-early strength concrete slab was examined experimentally by Fam and Skutezky (2006). The tubes were connected with GFRP dowels to transfer the shear. Some of the tubes were filled with concrete and some were not. One of the beams had CFRP laminate attached to the bottom flange to increase the flexural capacity ([Figure 3](#)). The beam with the CFRP layer was 31% stronger in flexure than the one without this layer. However, the authors found that the increase in stiffness was insignificant due to the very small CFRP reinforcement ratio. The beam failed by fracture of the CFRP laminate, followed immediately by fracture of the GFRP flange due to the sudden decrease in the section capacity under the high loading. The pseudo-ductility and progressive failure observed by Deskovic et al. (1995b) did not occur in this case.

Composite action between high performance materials requires good bonding between the components. Of particular interest for the current project is the bond between GFRP and UHPC, and the bond between the SRP and the GFRP section. A review of the research on the bond between FRP and other materials revealed that many researchers have studied the bond experimentally and/or analytically. For example, Shao et al., (2005) investigated the bond between dry FRP laminates and wet cast-in-place concrete through several pullout tests. They compared this “wet” bond behavior with the more conventional “dry” bond – that between FRP and matured dry concrete. The results from the two bonding cases were very similar, and the authors suggested that appropriate bond can be achieved by casting concrete directly onto composite FRPs through an epoxy adhesive. Camli and Binici (2007) tested the strength of CFRP sheets bonded to concrete prisms and hollow clay tiles using a double shear push-out test setup. The tests revealed that embedded anchors acting as shear connectors doubled the bonding strength. The results also showed that the bond strength is more affected by the anchorage length and width than by the concrete compressive strength. When the results of tests on specimens with and without plaster were compared, the presence of a plaster finish was observed to decrease the bond strength by two-thirds. Finally, a slight improvement was noticed in the bond strength when the ends of the CFRP sheets were spread up to 30°. The bond strength of concrete to E-glass/Kevlar FRP plates was determined through double shear and bending tests, and from the results, Xiao et al. (2004) suggested that the effective bond length for a direct shear test is 100 mm and for flexural bond of the FRP plate in the negative moment zone is about 300 mm. Garden (2003) reported on the economic use of composites with concrete in construction and the results tests on 18.0 m (60.4 ft) full-scale hybrid FRP-concrete beams. The beams were 350×460mm (13.8 x 18.1 in.) constructed from glass- and carbon fibre prepegs laminated in separate layers. The web stiffeners were half-rounds of timber laminated over with a single glass-fibre layer to prevent web buckling. A 150×500 mm (6 x 19.7 in.) layer of concrete was cast on the top of the beam. The GFRP-CFRP beam weighed 150 kg (331 lbs) without the concrete. The mode of failure was mainly cohesive fracture of the concrete near the sides of the beam, although localized adhesive failure between the concrete and composite were found before the beam collapsed suddenly probably due to buckling of the side panel. The author reported that buckling of the side panel in the shear spans was observed by a change in light reflection from the resin surface of the composite as the panels lozenged and caused the onset of the beam failure.

Zhou et al. (2005) examined the performance of cellular FRP bridge deck systems by studying the deck stiffness, strength and failure mode using two methods of deck contact loading: a steel patch according to the AASHTO Bridge Design Specifications and a simulated tire patch constructed from an actual truck tire. The authors found that the simulated tire loading developed greater global deflections under static loading, and the failure mode was localized and dominated by transverse bending failure of the composites, as opposed to the punching shear observed for the steel patch load. The authors conducted field performance tests on the bridge deck under service conditions and found no significant loss of deck capacity after more than one year.

On the theoretical side, an analytical model for defining nonlinear bond stress-slip models of FRP-concrete interfaces was proposed by Jianguo et al. (2005). Pullout tests were also conducted with different FRP materials and stiffnesses, and different adhesives. The results of the analytical models showed good agreement with the experimental data. Ferracuti et al. (2007) developed a procedure for deriving the non-linear mode II interface law for FRP-concrete bonding based on existing experimental results. The formula included the non-linear contribution of the adhesive and concrete cover at high shear stresses considering pull-pull delamination tests. The authors performed a numerical simulation for a bond-slip model, and obtained good agreement with the experimental results for the FRP strains, shear stresses and slips in the bonded region. Closed-form and finite-difference solutions for determining the elastic shear and peel stresses in the bonding zone between a strengthening plate and a beam were presented by Stratford and Cadei (2006). The methods consider the loading and temperature effects in addition to plate prestrains, and can be used in designing a strengthening scheme for a beam. The methods allow for a change of plate area along the beam length, and can determine the sensitivity of an adhesive joint to bond defects. Yuan et al. (2004) derived a closed-form analytical expression for the interfacial shear stress distribution and load-displacement response for different loading stages. Oehlers et al. (2003) suggested different structural detailing approaches to be considered in order to avoid debonding mechanisms.

EXPERIMENTAL WORK

Materials

The materials used in the work described here are: unidirectional carbon fiber-reinforced polymer (CFRP) sheets, steel reinforced polymer (SRP) strips, ultra-high-performance concrete (UHPC), a pultruded glass fiber-reinforced polymer (GFRP) box section, GFRP shear studs, moisture insensitive high modulus epoxy, and moisture tolerant epoxy. The stress-strain behaviors of the materials are presented in [Figure 4](#).

According to the manufacturer, the ultimate tensile strength of the CFRP sheets is 3450 MPa (500 ksi), the Modulus of Elasticity of the material is 230 GPa (33,359 ksi) and the elongation at failure is greater than 1.5%. The average tensile strength of a cured CFRP laminate with a moisture tolerant epoxy is 894 MPa (130 ksi) and the average elongation at failure is 1.33% (Sika 2007a).

The SRP consists of brass-coated, unidirectional, ultra-high-strength, twisted steel wires forming cords, which are assembled into a sheet that can be impregnated using various resins. The SRP system is similar in concept to externally-bonded FRP materials, which have been successfully implemented in many projects worldwide. The difference is that high-strength steel wires are used instead of carbon fibers as in CFRP composites. The specific SRP used was formed from 20 aligned cords per 25.4 mm (1 in.) with each cord consisting of three 0.35-mm (0.014 in.) diameter straight filaments surrounded by two filaments at a high angle of twist (Hardwire™ 3×2-20 Tape). The cord has a breaking load of 1539 N (346 lb) (confirmed with ASTM D 3039-00 tensile test), and Modulus of Elasticity of 160 GPa (23,206 ksi). The effective Modulus for the sheet is 66 GPa (9572 ksi) (Hardwire LLC 2007). The SRP's cords are assembled beside each other during the manufacturing process with a silicon mesh on one side. The properties of the CFRP sheet and the SRP strip are presented in [Table 1](#).

The constituent materials formulated to make the UHPC include Portland cement, silica fume, quartz flour, fine silica sand, a high range water reducer, water and steel or organic fibers (Lafarge 2007). Some of the available UHPC products on the market can reach compressive strengths of up to 200 MPa (29 ksi) and flexural tensile strengths of up to 50 MPa (7.25 ksi). In order to reach maximum strength, the UHPC should be heat-treated for a few days. Here, the samples were not treated thermally as that would have affected the bonding efficiency of the epoxy. The UHPC type used had steel fibers and reached a compressive strength of 170 MPa (24,656 ksi) after 28 days, about 75% of the strength which could have been achieved with thermal treatment. The presence of the fibers

within the concrete plays a major role in reducing the shrinkage strains within the UHPC. The distribution of the fibers within the concrete also increased the fatigue endurance limits of the UHPC significantly as reported by Farhat et al. (2007). Shaheen and Shrive (2007) also conducted experimental fatigue test on UHPC beams: the results showed that the material has an excellent resistance to low and high cyclic fatigue load even in the presence of a crack. Also the fatigue tests performed by Parsekian et al. (2008) on a UHPC bridge girder showed high fatigue resistance without observed degradation in the stiffness of the girder.

Based on the ASTM tests conducted by the manufacturer (Strongwell 2007), the minimum ultimate mechanical properties for the GFRP hollow box beams are as follows: the longitudinal tensile and compressive stresses are 207 MPa (30 ksi); the transverse tensile and compressive stresses are 48.3 MPa (7 ksi) and 103 MPa (15 ksi) respectively; the longitudinal tensile and compressive modulus are 17,200 MPa (2495 ksi); and the transverse tensile and compressive modulus are 5.52 GPa (800 ksi) and 6.89 GPa (999 ksi), respectively. Tests in single shear on the 9.5 mm (3/8 in.) diameter GFRP studs showed that the average capacity in single shear is 10,000 N (2248 lb). In addition to the structural contribution of the GFRP box section, the section will act as a permanent, stay-in-place form for the concrete.

Two types of epoxies were used - Sikadur[®] 32 HI-MOD and Sikadur[®] 330. According to the manufacturer (Sika 2007b, c), the Sikadur[®] 32 moisture-insensitive epoxy adhesive had a pot life of 30 minutes, tensile strength of 33 MPa (4.79 ksi) and modulus of elasticity of 3030 MPa (439 ksi). The bond strength of this epoxy when joining plastic to hardened concrete reached 13 MPa (1.89 ksi), and the same strength was achieved for binding plastic concrete to steel surfaces. The Sikadur[®] 330 moisture-tolerant epoxy adhesive had a pot life of 30 minutes at room temperature, tensile strength of 30 MPa (4.35 ksi) and modulus of elasticity of 3800 MPa (551 ksi).

Description of the proposed profile

All the materials used in the tests have superior strength and durability compared to conventional construction materials. They can be utilized in much smaller dimensions than traditional sections. A proposed section (as shown in [Figure 5](#)) has been developed to satisfy ease of construction with minimum production cost. The hollow GFRP section has outer dimensions of 152.4 mm × 228.6 mm (6 × 9 in), and a constant wall thickness of 11.11 mm (7/16 in.) (Strongwell 2007). The used GFRP box section in this study was selected from available commercial sections over the shelf, so the box dimensions do not utilize the materials as efficiently as could be achieved. At this stage, however, the objective of the study was to investigate the flexural behavior of the hybrid section, as well the interaction and the compatibility between the high strength materials at service and ultimate loads. On one beam, a 127 mm (5 in) width SRP strip was bonded to the lower side of the GFRP section to carry the tensile stresses. On the other beam, two layers of CFRP sheets were bonded to the bottom flange, each 127 mm (5 in.) wide. A 54 mm (2-1/8 in.) layer of UHPC was placed on top of the section to carry the compressive stresses and to support the upper GFRP flange.

The lower SRP sheet was bonded to the GFRP section by a moisture tolerant epoxy (Sikadur[®] 330), while the wet UHPC was bonded to the GFRP section with the moisture insensitive epoxy (Sikadur[®] 32). Other alternatives for joining the UHPC and the GFRP section may include using FRP studs, steel studs, irregular winding FRPs, interlocking some of the UHPC by holes in the GFRP upper flange, or some combination of these methods. Here, 9.5 mm (3/8") GFRP studs were used in conjunction with the moisture insensitive epoxy. Good bond strength was achieved during the experiments. The epoxy between the GFRP box section and the UHPC also worked as an insulating layer to protect the FRP section from any possible chemical attack from the concrete.

The possible failure modes considered for the proposed section included compression failure of the UHPC, tensile failure in the SRP, shear failure of the GFRP web, debonding of the SRP or UHPC, GFRP web buckling due to shear, and lateral torsional buckling of the beam.

Section assembly

Two beams were designed and tested considering the constructability of the section and taking into consideration the need for shear transfer between the GFRP box section and the UHPC. The hollow box beam sections have a smooth and shiny finish, so the top and bottom surfaces were sanded in preparation for bonding. A Hardwire SRP sheet was bonded to the bottom of one beam and two CFRP sheets were bonded to the bottom of the other. Holes were drilled

in the top flange of both beams and GFRP studs were bonded to the box section with the moisture tolerant epoxy Sikadur® 330, the same type used for bonding the CFRP sheets and SRP to the bottom flange. A thin layer 54 mm (2-1/8 in.) of wet UHPC was placed on top of each beam (Figure 6) and bonded to the beam with the moisture insensitive epoxy.

Experimental tests

From previous experiments reported in the literature, it was determined that using epoxy only for bonding the concrete to the GFRP sections would not provide sufficient strength. Debonding failure occurred at stresses much lower than the expected bonding strength, which suggested that epoxy discontinuity in the bonding area might have caused Mode II fracture failure. The discontinuities act as flaws which initiate cracking at low stresses. In order to ensure the efficiency of the bonding agents, preliminary bonding tests were performed on the materials used. Two sets of experimental tests were completed to investigate the efficiency of the bond between the different materials to be used in the proposed beams. The first set was concerned with the efficiency of bonding wet UHPC to FRP and SRP using the moisture insensitive epoxy (Elmahdy et al. 2007b). The second set measured the bond strength of the SRP and FRP using the moisture insensitive epoxy and the moisture tolerant epoxy (Elmahdy et al. 2007a). The results of these experiments revealed that the bonding strength between the elements would be enough for the main experimental configuration.

The middle sections of the two beams were instrumented with strain gauges and Linear Strain Conversion Transducers (LSCT) to measure the longitudinal strains. Additionally, two string potentiometers were used to measure the deflection at mid span, and LSCTs were placed perpendicular to the side of the beams to measure any lateral translation as shown in Figure 7.

The beams were tested in four-point loading with a 1.20 m (3.93 ft) spacing between the loading points as shown in Figure 8. The loading configuration for Beams 1 and 2 is shown in Figure 9. These beams were tested 28 days after the UHPC had been cast. The dimensions of the steel bearing plates used at the supports and under the loading points affected the failure modes of the beams: the dimensions of the steel plates for each beam are provided in Table 2.

Experimental results

The first beam tested (Beam 1) had one layer of SRP sheet as the tension reinforcement. The relatively thin bearing plate of Beam 1 (Figure 10) caused stress concentration at the supports and resulted in failure through crippling at the support (Figure 11) under a load of about 80% of the estimated section capacity. Thicker bearing plates were used for Beam 2 to avoid the crippling failure (Figure 12). Beam 2 failed by crushing/shear failure of the UHPC under one of the loading points (Figure 13), followed by delamination between the concrete and the GFRP box section. The failure occurred at the section where maximum shear and normal stresses developed, at the middle of the loading plate.

Beam 1 was retested by testing the beam over a shorter span avoiding the damaged end. The shorter beam is referred to as Beam 3. Thicker steel plates were used at supports and loading points of Beam 3 (Figure 14), as it was expected that the failure load would be higher due to the shorter span. Beam 3 failed by crushing of the UHPC under one of the loading points like Beam 2 (Figure 15), but because a thicker loading plate was used, the failure occurred at the end of this plate. The results for the three beams are summarized in Table 3.

The behavior of the beams was linear elastic up to failure. After Beam 3 had been loaded to 242 kN (about 95% of the failure load), it was unloaded so the load cell could be changed: at zero load, the beam had a residual deflection of only 5.5 mm (0.22 in), less than 3.5% of the maximum measured deflection. The strain distributions measured along the sections of Beams 1 and 2 at different load levels are shown in Figure 16. It is clear that the strain distribution is linear, and the location of the neutral axis is almost constant.

Due to the fine nature of the UHPC constituents, the strain gauges worked up to loads close to the failure loads in most locations. No cracks appear in the UHPC up to the failure, but some spalling was noticed on the top surface of the concrete prior to failure. The upper surface layer of the UHPC (1 mm [0.04 in.]) had some voids, and this layer peeled at intermediate to high loads. Hence if a strain gauge is to be bonded to this surface, the surface should be sanded so that the strain gauges will be bonded to more solid material.

Adding the UHPC and SRP sheet to the GFRP box section increased the flexural capacity of the box section by about 3.7 times that of the GFRP box alone. The shear stress between the UHPC and the GFRP box section reached five times the capacity of the shear studs alone, confirming the efficiency of the moisture insensitive epoxy.

Beams 1 and 2 were initially designed to have similar capacity, and both beams behaved similarly during the tests. The load-deflection relationships for both beams are shown in [Figure 17](#); Beam 2 was slightly more flexible than Beam 1. The lateral displacements measured for the three beams were small. The lateral displacement for Beam 1 was 2.84 mm (0.11 in.) at a load of 130 kN (29.2 kip), while the lateral displacement for Beam 2 was 0.33 mm (0.01 in.) at the same load. Thus, the buckling of the side panels which was observed by Garden (2003) when loading a GFRP beam with normal concrete on top and CFRP to resist the tension, appeared not to be occurring. Later, the design section will be optimized to use the materials more efficiently and obtain some warning from the section before failure.

ANALYTIC WORK

A simple method of analysis was used to predict the behavior of the hybrid section. The main assumptions are:

- Transverse plane sections remain plane after bending.
- Full bond occurs between all elements.
- The effect of the shear studs and the holes drilled in the top flange of the GFRP box can be neglected.
- The failure of the section can be predicted from the maximum-strain failure criterion.
- The contribution of the epoxy between the UHPC and the GFRP hollow box can be neglected.
- The UHPC is not cracked due to shrinkage.
- The shear stresses are uniform across the width of the section.
- All the materials have linear stress-strain relationships.
- The fillets of the GFRP section can be ignored.

A linear strain distribution is assumed in the simplified analysis, with the centroid of the section being located through enforcing internal equilibrium of the tension and compression forces within the section. A visual basic subroutine was used to determine the internal forces and moments in the section at different strain levels. The contribution to the overall stiffness of the hybrid section by each material is given in [Table 4](#). The graphs in [Figure 18](#) compare the strains measured at the top and bottom of the three beams with those calculated by the simplified analysis technique. The mechanical properties of the SRP and UHPC were determined from experiments on samples of the material. SRP samples were tested in tension, and cylinders of UHPC prepared during casting of the UHPC were tested in compression at an age of 28 days. A plain GFRP beam was loaded up to 10 kN (2248 lb) and its linear load-deflection relation was used to calculate the Young's modulus of the GFRP.

A 3-D linear finite element modeling for the 6.0 meters (19.67 ft) beams was done using ABAQUS 6.7 code, the models were built to represent the actual beam dimensions and materials (Elmahdy et al. 2008). The linear analysis was used because of the almost linear stress-strain relationships for all the materials. Both the SRP and CFRP sheets were modelled using 4-node linear quadrilateral shell elements, while 8-node linear hexahedron brick elements were used to model the GFRP box section and the UHPC. The finite element results confirmed the analytical study which is presented here. Good agreement in the location of the neutral axes, the linear strain distribution along the section height, the locations of maximum stresses, and the bulging of the GFRP box section at the supports were achieved. The accurate analytical and finite element calculations for the beam performance indicate the possibility of predicting the flexural behaviour of the proposed hybrid FRP-concrete section by linear analysis.

CONCLUSIONS

The following conclusions can be drawn from the results:

- Adding UHPC and (SRP or CFRP) can increase the capacity of GFRP box sections significantly.

22 Elmahdy et al.

- The bonding of wet UHPC to the GFRP box section can be affected to satisfy the requirement of many projects.
- The proposed beam assembly showed acceptable lateral torsional stability.
- Strain was distributed linearly through the depth of the hybrid beam up to failure.
- The composite beams failed suddenly without any ductility.
- A simple analysis technique can predict the flexural behavior of the beam.
- Strain gauges can function on UHPC and record accurate readings.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support of the Alberta Ingenuity Fund and the Natural Science and Engineering Research Council of Canada for the presentation of this work, and Hardwire LLC, Strongwell, Lafarge North America, and Sika Canada Inc for providing the materials used in this research. The dedication and help of the technical staff of the Department of Civil Engineering is gratefully recognized.

REFERENCES

- Aref, A. J., Kitane, Y., and Lee, G. C. (2005). "Analysis of Hybrid FRP-Concrete Multi-Cell Bridge Superstructure," *Composite Structures*, 69(3), pp. 346-359.
- Camli, U. S., and Binici, B. (2007). "Strength of Carbon Fiber Reinforced Polymers Bonded to Concrete and Masonry," *Construction and Building Materials*, 21(7), pp. 1431-1446.
- Deskovic, N., Meier, U., and Triantafillou, T. C. (1995a). "Innovative Design of FRP Combined with Concrete: Long-Term Behavior," *Journal of Structural Engineering*, 121(7), pp. 1079-1089.
- Deskovic, N., Triantafillou, T. C., and Meier, U. (1995b). "Innovative Design of FRP Combined with Concrete: Short-Term Behavior," *Journal of Structural Engineering*, 121(7), pp. 1069-1078.
- Elmahdy, A., El-Hacha, R., and Shrive, N. (2007a) "Bonding Behaviour between CFRP Plates, SRP Sheets, and Concrete," *Proceedings of the Twelfth International Colloquium on Structural and Geotechnical Engineering (12th ICSGE)*, Cairo, Egypt. December 10-12, 2007, (CD-ROM, 10p.)
- Elmahdy, A., El-Hacha, R., and Shrive, N. (2007b) "Bonding of CFRP and SRP to Ultra-High Performance Concrete," *Proceedings of the The First Asia-Pacific Conference on FRP in Structures (APFIS 2007)*, Hong Kong. December 12-15, 2007, (CD-ROM, 8p.)
- Elmahdy, A., Shrive, N., and El-Hacha, R. (2008) "Finite Element Analysis of Hybrid FRP-Concrete Composite Girders," *Proceedings of the The Fifth International Conference on Advanced Composite Materials in Bridges and Structures (ACMBS-V)*, Winnipeg, Manitoba, Canada. September 22-24, 2008, (CD-ROM, 10p.)
- Fam, A., and Skutezky, T. (2006). "Composite T-Beams Using Reduced-Scale Rectangular FRP Tubes and Concrete Slabs," *Journal of Composites for Construction*, 10(2), pp. 172-181.
- Farhat, F. A., Nicolaidis, D., Kanellopoulos, A., and Karihaloo, B. L. (2007). "High Performance Fibre-Reinforced Cementitious Composite (CARDIFRC) - Performance and Application to Retrofitting," *Engineering Fracture Mechanics*, 74(1-2), pp. 151-167.
- Ferracuti, B., Savoia, M., and Mazzotti, C. (2007). "Interface Law for FRP-Concrete Delamination," *Composite Structures*, 80(4), pp. 523-531.
- Garden, H. (2004). "Use of Advanced Composites in Civil Engineering Infrastructure," *Structures & Buildings*, 157(6), pp. 357-368 (SB6 buildings and structures).

Hardwire LLC. (2007). "3X2 Cord and Tape Specification Sheet." <http://hardwirellc.com/>, accessed on June 8, 2008.

Jianguo, D., Tamon, U., and Yasuhiko, S. (2005). "Development of the Nonlinear Bond Stress--Slip Model of Fiber Reinforced Plastics Sheet--Concrete Interfaces with a Simple Method," *Journal of Composites for Construction*, 9(1), pp. 52-62.

Lafarge. (2007). "Ductal - Technical Characteristics."

Oehlers, D. J., Park, S. M., and Mohamed Ali, M. S. (2003). "A Structural Engineering Approach to Adhesive Bonding Longitudinal Plates to RC Beams and Slabs," *Composites Part A: Applied Science and Manufacturing*, 34(9), pp. 887-897.

Parsekian, G., Shrive, N., Brown, T., Kroman, J., Perry, V., and Boucher, A. (2008). "Static and Fatigue Tests on Ductal UHPFRC Footbridge Sections," *ACI Special Publication of the Fifth International ACI/CANMET Conference on High Performance Concrete Structures and Materials, HPC 2008, Manaus, Brazil, June 18, 2008 (submitted)*.

Shaheen, E., and Shrive, N. (2007). "Cyclic Loading and Fracture Mechanics of Ductal Concrete," *International Journal of Fracture*, 148(3), pp. 251-260.

Shao, Y., Wu, Z. S., and Bian, J. "Wet-Bonding between FRP Laminates and Cast-In-Place Concrete," *Proceedings of the International Symposium on Bond Behaviour of FRP in Structures*, Hong Kong, China. December 8-10, 2005, pp. 91-96.

Sika. (2007a). "Product Data Sheet – Sika Carbodur."

Sika. (2007b). "Product Data Sheet – Sikadur 32 Hi-Mod."

Sika. (2007c). "Product Data Sheet – Sikadur 330."

Stratford, T., and Cadei, J. (2006). "Elastic Analysis of Adhesion Stresses for the Design of a Strengthening Plate Bonded to a Beam," *Construction and Building Materials*, 20(1-2), pp. 34-45.

Strongwell. (2007). "Metric Design Information."

Xiao, J., Li, J., and Zha, Q. (2004). "Experimental Study on Bond Behavior between FRP and Concrete," *Construction and Building Materials*, 18(10), pp. 745-752.

Yuan, H., Teng, J. G., Seracino, R., Wu, Z. S., and Yao, J. (2004). "Full-Range Behavior of FRP-to-Concrete Bonded Joints," *Engineering Structures*, 26(5), pp. 553-565.

Table 1 — FRP material properties as reported by the manufacturers

FRP products	Dimensions, mm (in.)	Elastic Modulus, MPa (ksi)	Ultimate Tensile Strength, MPa (ksi)	Reference
Unidirectional CFRP Sheet (SikaWrap Hex 230C with Sikadur 330 Epoxy)	0.381 (0.015) [‡]	65,402 (9486)	894 (130)	Sika (2007)
Unidirectional SRP Sheet (Hardwire™ 3×2-20-12)	1.23 (0.048) [‡] 0.38 (0.015) [†]	66,177 (9598)	985 (143)	Hardwire (2007)

[‡] Thickness [†] Net area per width mm²/mm (in.²/in.)

Table 2 — Dimensions of the steel plates used at the supports and loading points

Beam	Span, m (in.)	Bottom Sheet	Support plate dimension		Loading plate	
			Thickness mm (in.)	Width mm (in.)	Thickness mm (in.)	Width mm (in.)
1	6.00 (236)	SRP	12.7 (0.5)	101.6 (4)	12.7 (0.5)	101.6 (4)
2	6.00 (236)	CFRP	50.8 (2)	101.6 (4)	12.7 (0.5)	101.6 (4)
3	5.10 (200)	SRP	63.5 (2.5)	203.2 (8)	38 (1.5)	101.6 (4)

Table 3 — Summary of results for all beams

Beam	Maximum load, kN (kip)	Maximum deflection, mm (in.)	Maximum moment, kN·m (kip·ft)	Maximum shear, kN (kip)
1	165.7 (37.25)	167.1 (6.58)	199 (146.8)	82.9 (18.6)
2	193.6 (43.52)	212.5 (8.37)	232 (171.1)	96.8 (21.76)
3	257.5 (57.89)	165.8 (6.53)	251 (185)	128.8 (28.96)

Table 4 — The percent contribution of the different elements to the overall section stiffness

	Beam 1			Beam 2		
	UHPC	GFRP	SRP	UHPC	GFRP	CFRP
Contribution Percentage (%)	24.6	62.2	13.2	24.3	65.9	9.8

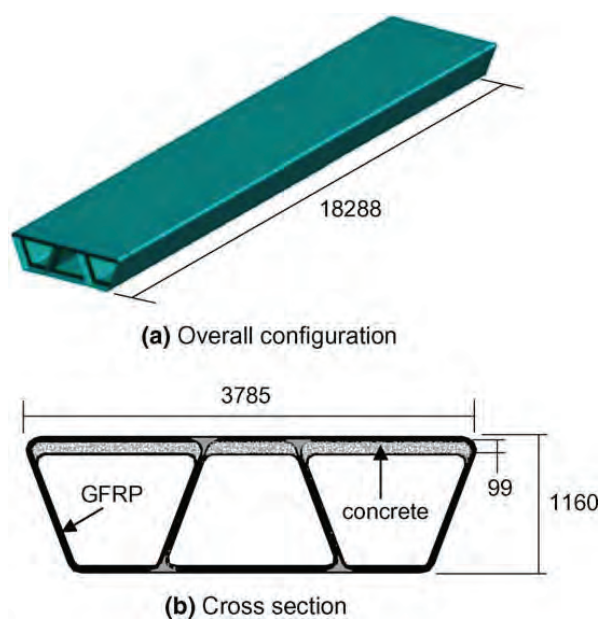


Figure 1 — FRP-Concrete bridge (Aref et al. 2005)

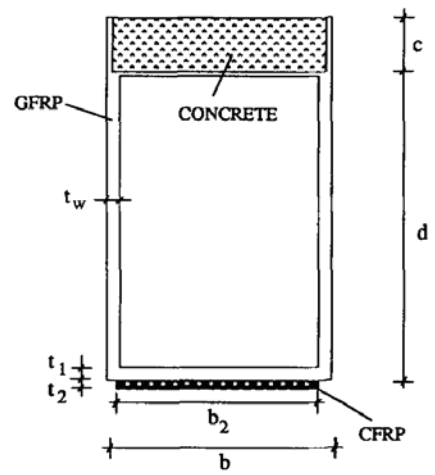


Figure 2 — Hybrid section (Deskovic et al. 1995b)



Figure 3 — Concrete slab connected to GFRP tubes (Fam and Skutezky 2006)

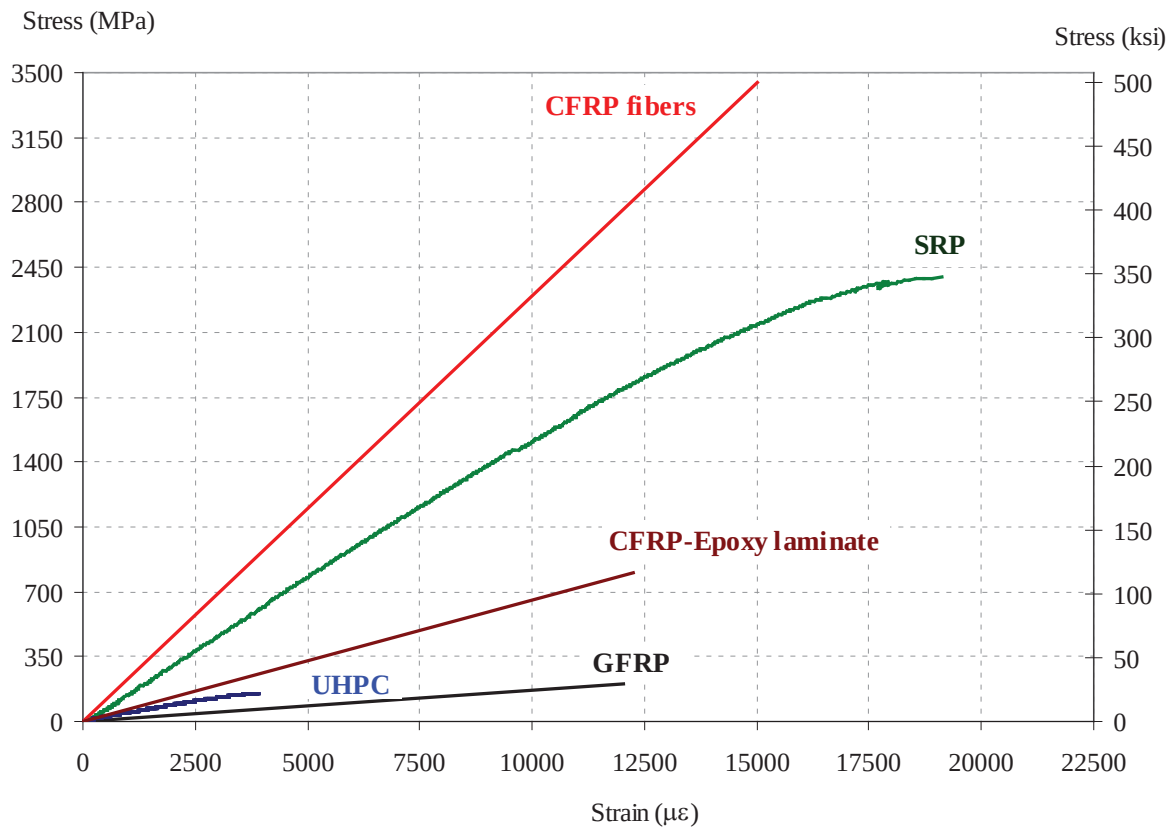


Figure 4 — Stress-strain relationships for the materials of the hybrid sections

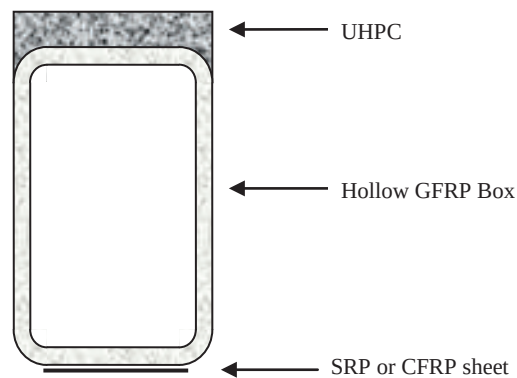


Figure 5 — The proposed hybrid FRP-concrete cross section

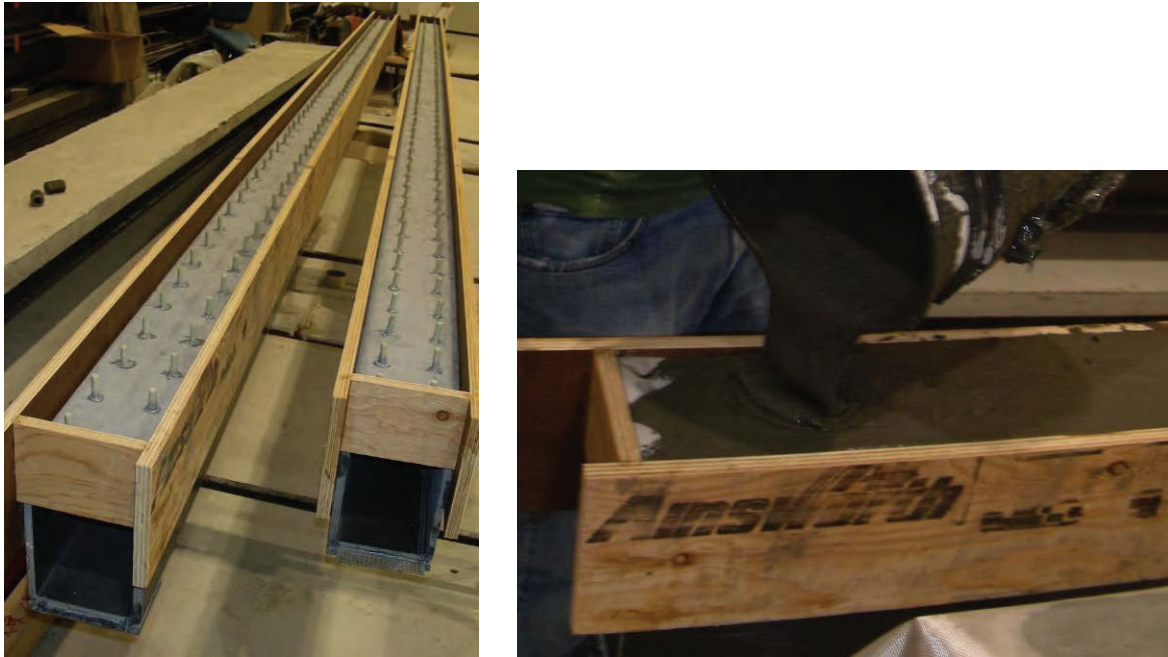


Figure 6 — The UHPC wood form before and during casting of the UHPC



Figure 7 — Instrumentation of the beams at the mid-section



Figure 8 — Test arrangement for Beam 1

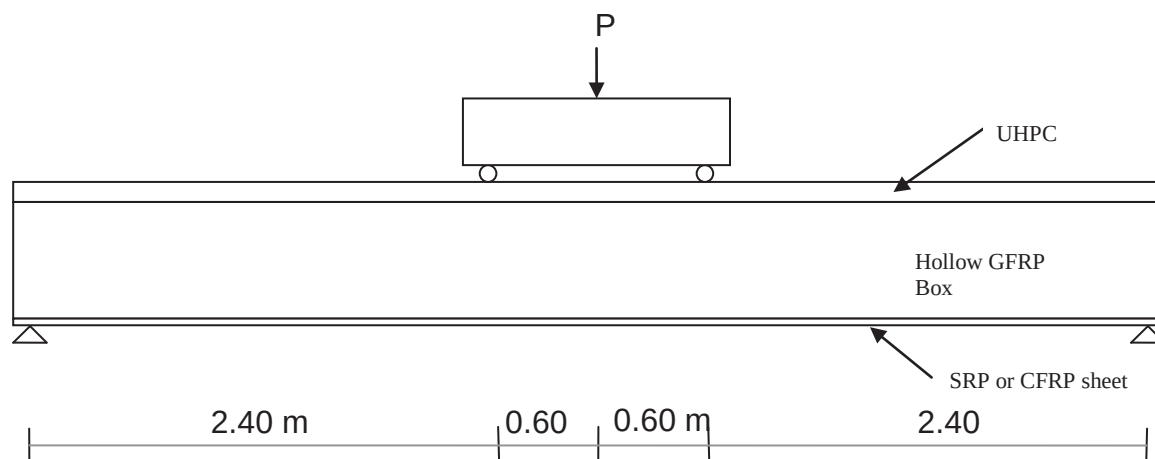


Figure 9 — Loading configuration for Tests 1 and 2



Figure 10 — Bearing plate support of Beam 1



Figure 11 — Failure of Beam 1

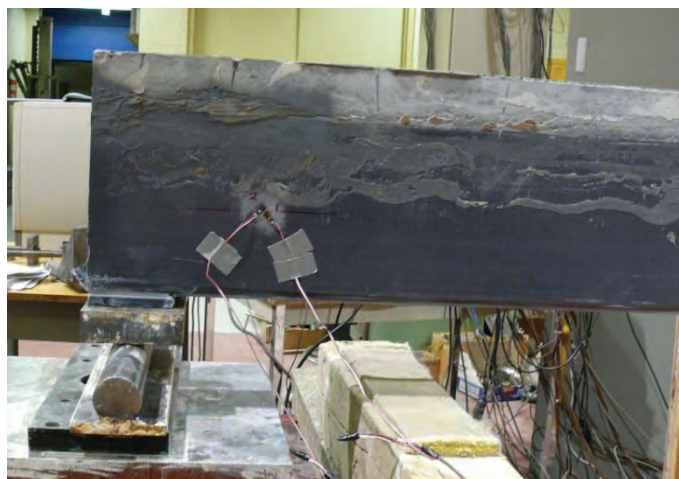


Figure 12 — Thick plate at the support of Beam 2



Figure 13 — Failure of Beam 2



Figure 14 — Loading plate for Beam 3



Figure 15 — Failure of Beam 3

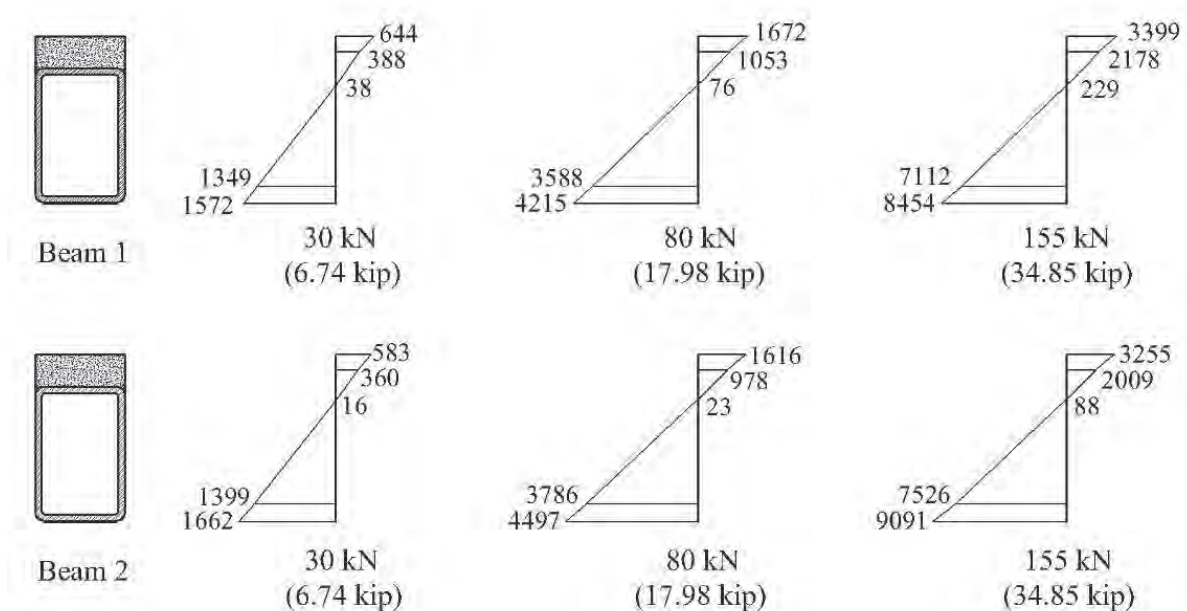


Figure 16 — Strain distributions at midspan for Beams 1 and 2 at different loads

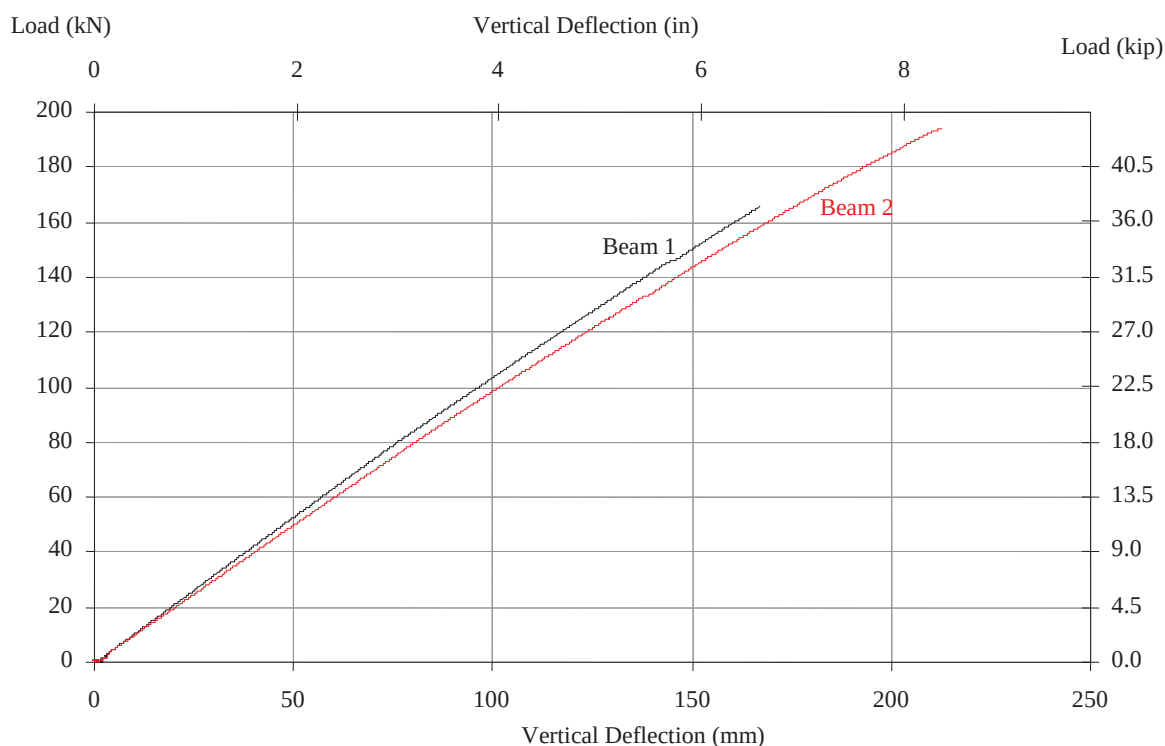


Figure 17 — Load versus midspan deflection for Beams 1 and 2

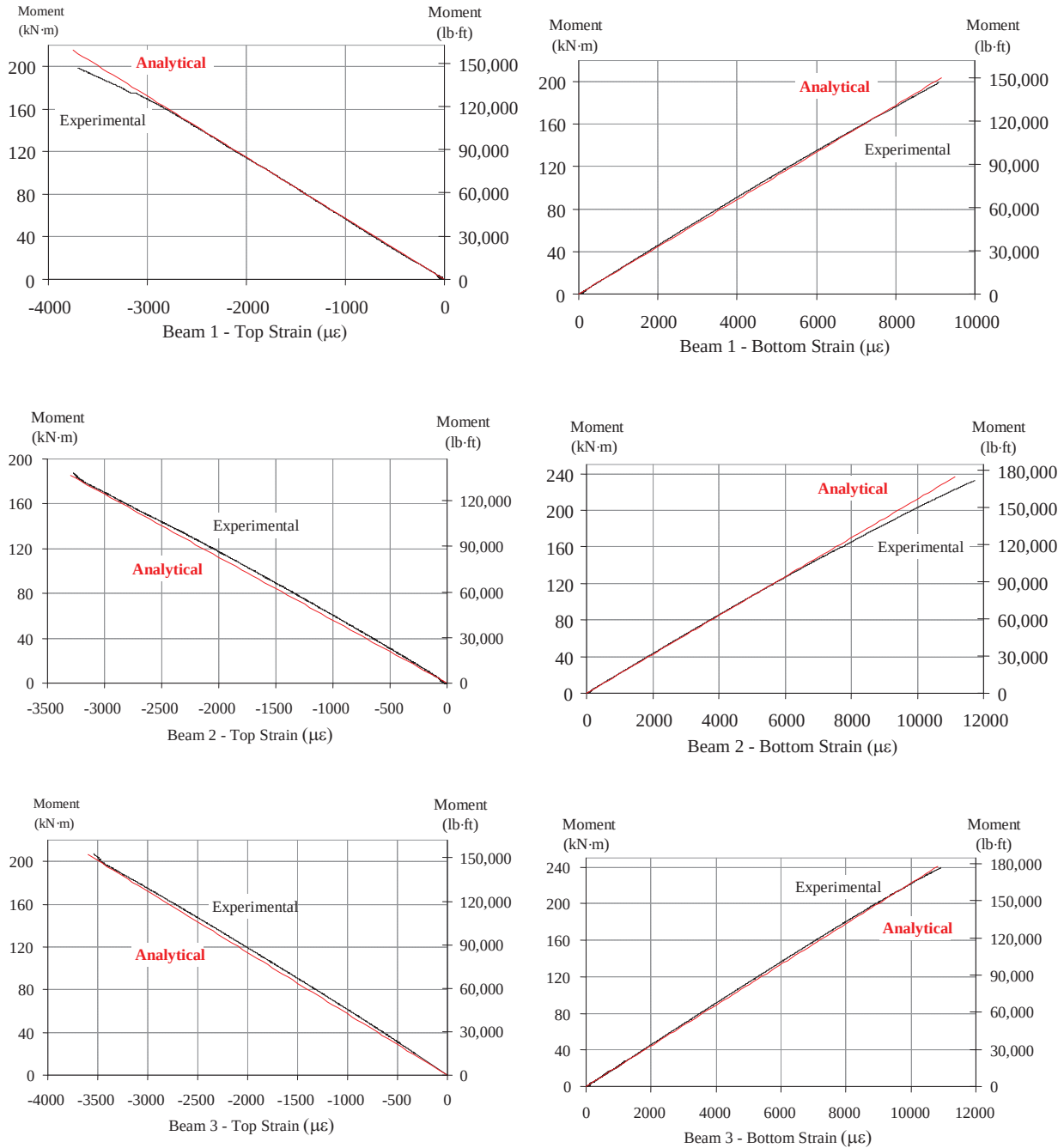


Figure 18 — Top and bottom strains for the beams at different loading moments

Wheel Load Distribution in Concrete Bridge Decks with FRP Stay-in-Place Forms

by C.M. Foley, B. Wan, and J. Liu

Synopsis: The State of Wisconsin recently constructed a highway bridge as part of the FHWA Innovative Bridge Research and Deployment Program. The superstructure is composed of precast concrete I-girders acting compositely with the concrete bridge deck that uses a fiber-reinforced polymer stay-in-place (FRP-SIP) formwork system that serves as positive flexural reinforcement. In-place load test strain data are used to compute wheel load distribution widths and estimates for bending moments acting on unit widths of bridge deck. These widths are compared with those computed using the AASHTO-LRFD and AASHTO Standard specifications. The in-place field data indicate that bending moments within the deck system are reasonably close to predictions made using the Standard and LRFD specification bridge deck approximate analysis procedures. The Standard specification estimates for bending moment per unit width of bridge deck were found to be conservative for interior and exterior deck spans. The LRFD specification estimates for strip width resisting wheel loads are also conservative for interior and exterior deck spans when compared to field-measurement-based predictions.

Keywords: bridge decks; fiber-reinforced polymer (FRP); load distribution width; stay-in-place (SIP); wheel load distribution

Christopher M. Foley, PhD, PE – Associate Professor of Civil Engineering at Marquette University in Milwaukee, Wisconsin. Dr. Foley has been teaching courses at Marquette University for the past 11 years including Bridge Design. He is a member of the AISC Specification Task Committee on Connections and is also Chair of the ASCE/SEI Technical Committee on Optimal Structural Design. He is PI of the WisDOT IBRC monitoring effort upon which the present manuscript is based.

Baolin Wan, PhD – Assistant Professor of Civil Engineering at Marquette University in Milwaukee, Wisconsin. He is a member of ACI and an associate member of ACI Committee 440, Fiber-Reinforced Polymer Reinforcement. He is co-PI of the WisDOT IBRC monitoring effort upon which the present manuscript is based.

Junshan Liu, MS – Graduate Research Assistant at Marquette University in Milwaukee, Wisconsin. He received his MS in civil engineering at McGill University in Montreal, Quebec, and is currently a PhD student at Marquette University.

INTRODUCTION

The Wisconsin Department of Transportation (WisDOT), along with the University of Wisconsin-Madison, structural engineering consultants and bridge construction contractors, took a significant step in the direction of formalizing the use of novel structural engineering systems for bridges when they successfully proposed and received funding through the Federal Highway Administration's Innovative Bridge Research and Construction (IBRC) Program (now the Innovative Bridge Research and Deployment program) to evaluate a novel bridge superstructure system. The goals of this program pertinent to the present research effort are to develop new, cost-effective innovative material applications in highway bridges and to develop engineering design criteria for innovative products and materials for use in highway bridges and structures. To meet these goals, WisDOT and the University of Wisconsin at Madison conducted experimental concept validation, material characterization and generated material specifications for a novel fiber-reinforced polymer (FRP) composite stay-in-place (SIP) formwork system used in bridge B-20-133 on U.S. Highway 151 in Waupun, Wisconsin (Bank et al. 2005; Berg 2004; Berg et al. 2004; Dieter et al. 2002b; Helmueller et al. 2002). The bridge is a typical precast, pre-tensioned two-span superstructure system (two simple spans at time of construction and two-continuous spans under live loading). An elevation of the in-service bridge and its location within the state of Wisconsin are shown in [Figure 1](#). Marquette University has been charged with monitoring this (and another) IBRC Bridge in Wisconsin over a five-year period. The monitoring program includes in-situ load testing at periodic intervals and field inspections to evaluate performance of the novel system compared to traditionally-constructed superstructures. The present manuscript describes a small portion of this effort related to quantifying the width of bridge deck that resists superimposed wheel loading and its comparison to design specification analysis methods. Changes in this width over time will be used as one metric to monitor the behavior of the bridge deck system. The present manuscript discusses the first of several planned (first completed) load tests.

The bridge deck consists of a permanent FRP-SIP formwork system, a two-way mat of FRP I-shaped pultruded bars, and three-piece orthogonally placed cross-rods. A mockup of the system is shown in [Figure 2](#). The FRP-SIP deck panel is intended to be positive flexural reinforcement for the concrete deck and the I-shaped bars are negative moment flexural reinforcement. The shear transfer mechanism enabling the FRP-SIP formwork panel to be positive flexural reinforcement is 1/4-inch diameter aggregate bonded to the panel using epoxy adhesive. Freeze-thaw testing (Helmueller et al. 2002; Martin 2006; Wan et al. 2007) indicates that the shear strength at the interface between the FRP-SIP is affected by freeze-thaw cycles, but the demand at the interface is also expected to be very low (Martin 2006). The composite deck panel utilizes E-glass fibers included in stitched fabrics, CSM, and unidirectional rovings (Berg 2004). The FRP-SIP deck panels are 7 foot, 10 inches long (2388 mm) and span in simple fashion between bridge girders spaced at 8 foot, 8 inches (2642 mm) on center. The I-shaped bars and three-piece cross-rod components are pultruded products that utilize E-glass fibers (Berg 2004). Continuity reinforcement in the superstructure facilitating two-span continuous behavior in the girders under traffic (live) loading is composed of ASLAN 100 GFRP bars. Exhaustive description of the bridge deck system and material specifications can be found elsewhere (Berg 2004).

RESEARCH SIGNIFICANCE

The use of FRP-SIP formwork is a relatively new addition to deck systems used in bridge construction. The system developed for bridge B-20-133 was extensively studied in the laboratory (Berg 2004; Dieter et al. 2002a), but previous in-situ load testing did not consider wheel load distribution widths (Bank et al. 2005). Continued development of FRP-SIP systems (Ringelstetter et al. 2006) suggests that structural engineers will require information related to the in-situ load transfer characteristics of these systems. Furthermore, application of existing specification methodologies for analyzing bridge decks using these novel systems needs to be evaluated. The objectives of the present manuscript are to briefly describe bridge deck analysis methodologies that are subsequently shown to be suitable for conducting structural analysis of the FRP-SIP deck system; discuss the load testing protocols and procedures used to measure strain within the in-situ system; and make recommendations with regard to the ability of currently accepted analysis methods in relation to conducting structural analysis of these deck systems for design.

LIVE LOAD MOMENT COMPUTATIONS

The distribution of wheel loading within a bridge deck is dependent on three major factors: (a) the deck span length; (b) the restraint characteristics at the ends of the deck span; and (c) the girder stiffness. Experimental evidence (Allen 1991; Batchelor et al. 1978; Beal 1982; Csagoly et al. 1978) indicates that there is a significant level of membrane arching action present in many bridge decks and flexural models may not be the most appropriate method to compute maximum positive and negative moment stresses within bridge decks. Furthermore, studies have indicated that the stiffness of the girders plays a vital role in limiting the transverse negative bending moments over the girders (Batchelor et al. 1978; Beal 1982; Cao et al. 1996; Cao and Shing 1999; Csagoly et al. 1978; Fang et al. 1990; Newmark 1949).

There have been many research efforts conducted since the 1930's to evaluate the state of stress within cast in place (CIP) concrete bridge decks. Experiments carried out on scale models (Beal 1982; Fang et al. 1990) and full-scale models (Fang et al. 1990) have been very valuable in understanding wheel load distribution and have led to validation of analytical procedures to predict stresses within the bridge deck. However, these previous efforts focused on isotropic reinforcement layouts and traditional cast-in-place construction. There has been no data generated to date that supports the validity of applying analysis methods developed for CIP construction to deck systems that utilize stay-in-place FRP formwork as positive moment reinforcement.

The methods for computing live load moments within bridge decks have changed very little from the first proposal formulated by Westergaard (1930). Further work by Newmark (1949) resulted in the basis for the Standard Specification design procedure for bridge deck analysis (AASHTO 2002). The procedure used is quite straightforward. A bridge deck live load moment (per unit width of slab) for a simple span condition is estimated using (AASHTO 2002),

$$M_{LLM} = P_w \left[\frac{S + 2}{32} \right] \quad (\text{units are lbs and feet with result being lb-ft/ft}) \quad (1)$$

where: P_w is the wheel loading (HS20 or HS15 using standard specification live load models); and S is the span of the bridge deck (conservatively can be taken as the spacing of the girders if flange widths are relatively small compared to the span). It should be noted that 1 lb-ft/ft is 4.448 N-m/m and equation (1) is taken from the U.S. customary units version of the specifications. Bridge decks are often continuous over multiple interior girders and therefore, the simple-span moment per unit width is modified by a continuity factor (AASHTO 2002),

$$M_{LLM} = 0.80 \cdot P_w \left[\frac{S + 2}{32} \right] \quad (2)$$

The positive and negative live load bending moments are taken to be the same within the bridge deck and this requires that the structural engineer provide equal mats of steel reinforcement in the top and bottom layers. It should also be noted that equations (1) and (2) make no distinction of interior and exterior deck span and therefore, are intended to be conservative for interior span bending moment estimations when compared to those estimates generated for exterior deck span conditions.

The procedure implied by equations (1) and (2) was felt to be over-simplistic by many researchers and experimental and analytical evidence showed that the flexibility of the girders within the superstructure system affects the state of stress within the deck (Batchelor et al. 1978; Beal 1982; Cao et al. 1996; Cao and Shing 1999; Csagoly et al. 1978; Fang et al. 1990) and that simplistic computations may over-estimate negative moment tensile stresses over the girders in the bridge deck. Nonetheless, equations (1) and (2) were used successfully for decades and the Load and Resistance Factor Design (LRFD) specifications included a significant departure from the former procedure.

The LRFD procedure for bridge decks is very similar to the long-standing distribution-factor procedure used for bridge girder analysis. The LRFD procedure includes definition of a strip width used to facilitate use of bending moments computed using one-dimensional analysis. In other words, the structural engineer conducts an analysis of the bridge deck assuming it is a one-dimensional continuous beam with movable wheel loads within traffic lanes and then converts the bending moments to unit-width quantities using the strip width. Two strip widths are defined in cast-in-place bridge deck slabs (AASHTO 2006),

$$\begin{aligned} &\text{Positive Moment Strip Width (inches: 1 inch = 25.4 mm)} \\ &SW^+ = 26.0 + 6.6 \cdot S \end{aligned} \quad (3)$$

$$\begin{aligned} &\text{Negative Moment Strip Width (inches)} \\ &SW^- = 48.0 + 3.0 \cdot S \end{aligned} \quad (4)$$

The LRFD procedure affords the structural engineer the opportunity to utilize statically indeterminate analysis models and therefore, there is the ability to generate more accurate analysis results through these models. The assumption that bending moment magnitudes be determined through structural analysis was a significant departure from earlier specification procedures. It should be noted that empirical analysis procedures are allowed for some deck systems. To the authors' knowledge, there is no experimental evidence that demonstrates in-situ bridge behavior of deck systems utilizing the FRP-SIP formwork systems is appropriately modeled using equations (3) and (4).

IN-SITU FIELD TESTING

The in-situ testing was designed to allow positive bending moment strains within the deck system to be measured on the soffit (bottom) of the FRP-SIP formwork. Low-modulus polymer carriers for strain gauges were used to create portable strain gauges for the project (Figure 3). These strain measuring devices were calibrated and were mounted to the underside of the FRP-SIP panels. Calibration of the sensors was a very elaborate process whereby strains measured at the surface of a flexural member were compared to those measured with the portable strain gauges. These strain gauges were bolted to the bridge deck soffit using wedge-type expansion anchors extending through the 0.3-inch (8 mm) FRP-SIP formwork sheet into the concrete deck. FEA indicated that there were strain concentrations that formed as a result of machining done to the polymer carrier (Figure 3 at top) and therefore, each strain sensor used in the effort included development of its own calibration factor. The calibration factor was developed using controlled flexural analysis experiments. Further details of the calibration process and development of the strain sensor are available elsewhere (Schneeman 2006).

The instrument array used in the testing is shown in Figure 4. Two sensor lines were utilized. The first is designated as TW1. This sensor array is located at the centerline of the exterior deck span. The second sensor array designated as TW2 is located along the centerline of the first interior span of the bridge deck. Detailed locations of the sensor arrays relative to the abutment face are shown in Figure 4. The superstructure cross-section is shown in Figure 5 and detailed dimensions of the parapet/barrier and precast girders are given.

Two loading protocols with a single calibrated tri-axle dump truck were run. The first was intended to facilitate recording of peak positive moment strains in the exterior deck span (Figure 6). The truck was slowly brought onto the superstructure and temporarily stopped at the first sensor location (W2). The truck then proceeded forward at a slow (controlled) pace with additional stops at locations corresponding to the remaining four strain sensors and one draw wire transducer (DWT). These stops were essentially 17.5 inches (444 mm) apart. The second loading protocol (Figure 7) was designed to allow peak positive moment strains in the FRP-SIP panels in the first interior deck span to be measured. The same creep-forward-with-stop protocol used previously was used again in this case. However, it should be noted that the last stop in Protocol 1 was over a DWT, while the first stop in Protocol 2 was over a DWT instrument. Photographs of the loading process and sensor stop locations are shown in Figure 8. A

calibrated (weighed) tri-axle dump truck was used for the load testing. The axle configuration, spacing, and wheel loads for the vehicle used are shown in [Figure 9](#).

Strains measured in the instrument array during the field testing are given [Figures 10 and 11](#). The ordinate in the graphs is simply a data point taken during the testing and loosely correlates with the time instant during the testing. Data shown in the ordinate interval 2,750 to 3,500 in [Figure 10](#) correspond to the wheel loading positioned above the DWT in Protocol 1. Data shown in the ordinate interval 800 to 1,500 in [Figure 11](#) correspond to wheel loading positioned above the DWT in Protocol 2. A baseline correction was first applied to the data. A moving average (20 points ahead and 20 points behind the data point in question) was utilized. The final step in the field data correction included application of a correction factor defined for each sensor by Schneeman (2006) to account for the expected difference in strain at the surface of the FRP-SIP panel and the location of the strain gauge within the polymer carrier.

COMPARISON TO AASHTO ANALYSIS METHODS

An influence surface along the TW1 and TW2 sensor arrays were generated using the data acquired during the load testing. Five strain sensors were used during the testing along each line. The truck wheel was targeted to stop directly above each of these sensors as shown in [Figure 12](#). The targeted stops were (in succession): W2, W1, M, E1, and E2. When the calibrated truck wheel was above W2, there were strain readings at the remaining four sensors. It was assumed that this data could be used to generate a symmetric (extrapolated) layout of four additional strain readings to locations with 17.5-inch (444 mm) intervals behind (or in front of) the truck wheel. As a result, five strain readings and four extrapolated symmetric strain readings were generated where applicable.

[Figure 9](#) indicates that the front and first-rear-tandem axle are separated by 195 inches (4953 mm). Thus, when the front wheel is over sensor E2, the first rear-tandem axle is nearly 10 feet (3048 mm) from sensor W2. An average strain value for each stop (the plateaus in [Figures 10 and 11](#)) were used as the basis for sensor readings. [Table 1](#) illustrates these average values obtained at each sensor location and symmetric extrapolated sensor location values. The bold font values are the strain readings taken in the field and the italicized-font values are the magnitudes at symmetric locations extrapolated from the measured data.

The truck motion can be mentally pictured using the data values in [Table 1](#). The exterior span can be considered as an illustrative example. When the front wheel is over sensor TW1-E2, the fifth row in the exterior span segment of the table is referenced. The strain reading for the front wheel at this location is then 75 microstrain. There are four strain sensor readings behind the front wheel that provide measured strain data. The four strain readings ahead of the front wheel are therefore, projected (extrapolated) assuming symmetry. When the front wheel is over sensor TW1-M, there are two strain sensors behind and in front of the wheel. As a result, no extrapolation can be done and projected readings at +/- 70 in. (1.8 m) and +/- 52.5 in. (1.3 m) are not available. In other words, full symmetry with 140 inches (3556 mm) of length adjacent to the wheel loading could only be obtained for cases where the front axle was over sensor W2 or E2 (first or last stop on sensors).

The strain readings were used to generate equivalent bending moment magnitudes assuming fully-composite behavior between the FRP-SIP formwork and the concrete deck. Preliminary FEA indicated that this is a reasonable assumption for service-level loads (Martin 2006). The strain readings at the bottom surface of the FRP-SIP formwork were converted to strains using Bernoulli beam theory assumptions,

$$M = \frac{E_{comp} \varepsilon I_{comp}}{y_{bot}} \quad (5)$$

E_{comp} is the modulus used for the transformed section assuming that the concrete material has been transformed to equivalent FRP material (5.73e6 psi, 39 GPa). The modular ratio used for the computations was 1.56. I_{comp} is the second moment of area for the composite cross-section (474.2 in⁴, 197×10⁶ mm⁴). Its magnitude was computed using a 17.5-inch (440 mm) width and data related to the FRP grid and the FRP-SIP formwork panels (Berg 2004; Dieter 2002; Dietsche 2002). The total height of the in-situ bridge deck is 8 inches (200 mm). ε is the

measured (or extrapolated) strain from Table 1 and y_{bot} is the distance from the centroid of the transformed composite deck cross-section to the bottom surface of the FRP-SIP formwork (4.09 in, 104 mm).

The strain readings from Table 1 are used to generate bending moment magnitudes acting in the vicinity of the strain sensor and these magnitudes are assumed to be an average across the 17.5-inch width adjacent to the strain sensor (see Table 2). The total positive moment resisted within the linear influence surface adjacent to the wheel loads is simply the summation of all moment magnitudes computed using the measured strains. These total moments, designated as M_T^+ are given in Table 3. The total moments are then divided by the linear distance defining the area of influence to generate an *average* positive bending moment per unit width. This average positive bending moment is given in Table 3 as well. It should be noted that length of influence for W2, W1, M, E1, E2 is 157.5 in. (4001 mm), 122.5 in. (3112 mm), 87.5 in. (2223 mm), 122.5 in. (3112 mm), and 157.5 in. (4001 mm), respectively.

The bending moments per unit width of length found in Table 3 can be compared to the positive bending moment that would be computed for a 10.93-kip (48.6 kN) wheel load using equation (2). This wheel load magnitude is approximately 37% greater than an HL-93 or HS20 truck loading. Table 3 illustrates the positive design live load moments that would result if the deck span was taken as 8-foot, 8-inches or 2642 mm (see Figure 5). As shown in the Table, the Standard specification (AASHTO 2002) methodology conservatively estimates the bending moment per foot of width that would be seen in the deck. As expected, the exterior span would control the positive bending moment magnitude used for design and the Standard specification procedure is more conservative for the interior span. One could use the Standard specification procedure (AASHTO 2002) to conservatively analyze the FRP-SIP bridge deck at load levels slightly above service-level loading using the standard specification procedure.

Developing a comparison of field acquired strip widths with that recommended in the LRFD methodology (AASHTO 2006) is a little bit more cumbersome. First of all, a structural analysis is required to determine the bending moments that would be present in a one-dimensional continuous beam model of the deck. The present work takes a relatively simplistic approach. The peak positive bending moment within the exterior span of the deck is determined with the assumption that the outside support is a roller, and the first interior support is fixed. This model is justified because there are two equal magnitude wheel loads placed in a nearly symmetric fashion on either side of the first interior girder (see Figure 6). Therefore, there will be very little tendency for the deck to rotate over this interior support. If this assumption is made, the maximum positive bending moment, M_{max}^+ , in the exterior span with a single concentrated load, P , at mid-span is;

$$M_{max}^+ = \frac{5}{32} \cdot P \cdot S \quad (6)$$

The maximum positive bending moment strip width, SW^+ , for the exterior span can then be computed as follows;

$$M_{max}^+ = \frac{5PS}{32} = \frac{5(P_w/SW^+)S}{32} \Rightarrow SW^+ = \frac{5P_w S}{32M_{max}^+} \quad (7)$$

In the case of the interior span, the process is the same, but the structural analysis bending moment is computed using,

$$M_{max}^+ = \frac{PS}{8} = \frac{(P_w/SW^+)S}{8} \quad (8)$$

as a result of the fixed-fixed end conditions (see Figure 6). The positive moment strip width for the interior span condition is therefore,

$$SW^+ = \frac{P_w S}{8M_{max}^+}$$

Strip widths computed using the measured strain data and comparison to the positive moment strip widths computed using equation (3) are given in Table 3. The average positive moment strip width, SW^+ , for the exterior span computed using the measured strains is approximately 126 inches (3.22 m). The positive moment strip width

computed using the LRFD specifications (AASHTO 2006) is 83.4 inches (2.12 m). The average positive bending moment strip width for the interior span condition computed using the field-measured strain magnitudes is larger indicating a greater width of bridge deck resisting wheel loads.

The Standard Specifications (AASHTO 2002) and LRFD Specifications (AASHTO 2006) assume that the predominant load-transfer mechanism in the bridge deck is flexure. Recent research efforts are indicating arching action as a major load-transfer mechanism in many bridge deck configurations. Assuming the concentrated wheel load is at mid-span results in a shear-span-to-depth ratio for the deck of approximately 6.5. This would justify flexural behavior as the dominant load transfer mechanism in the bridge deck and acceptance of Bernoulli beam theory as a model for deck behavior. However, the edge conditions on the slab strip perpendicular to traffic assumed in the development of equations (2), (3), and (4) are likely not seen in the real bridge. As a result, differences in the strip widths computed using measured strains are expected. However, the important item to note is that the AASHTO analysis procedures result in conservative estimates for bending moment on a per-foot basis. The width of slab assumed to resist wheel loading in the LRFD specifications is smaller than that computed using experimentally determined strains from the in-situ testing. Therefore, designers will be evaluating service-level and near service-level behavior in a conservative manner using the specifications as the bending moments on a per-foot basis for design will exceed those likely seen in the bridge.

The AASHTO LRFD design specifications (AASHTO 2006) also include an empirical (tabulated) procedure for conducting live load analysis for bridge decks of "usual" configuration. The conditions defining the typical configuration pertinent to comparisons with the values measured during the in-situ load test are (AASHTO 2006):

- The bridge deck is supported on parallel girders.
- Multiple presence factors and dynamic load allowances are included in the tabular values.
- The bridge deck supported on at least 3 girders.
- The width of the bridge deck is not less than 14 feet between centerlines of the exterior girders.
- The moments in the table are "upper-bounds" for the moments in the interior regions of the slab.

The empirical design method (AASHTO 2006) is applicable for the present bridge deck. However, several assumptions are required in order to make a comparison with the measured values discussed in this manuscript. First of all, an impact modifier equal to 1.33 will be assumed. Secondly, the spacing of the girders (8 ft 8 inches or 2.6 m) and the width of the bridge deck (39 feet or 11.9 m) indicate that one design lane will control the magnitudes of the bending moments present in the bridge deck. Therefore, a multiple-presence factor of 1.20 should be considered. Finally, the empirical procedure assumes that a 8-kip (35 kN) wheel load generates the bending moments. The wheel loading in the present in-situ test is 10.93 kips (49 kN).

The magnitude of the positive bending moment taken from the AASHTO empirical procedure table is 6.09 k-ft (8.3 kN-m) for an 8.67 foot (2.6 m) span. Accounting for the multiple presence factor, the impact modifier and the wheel load difference built into the tabular value results in an empirical estimate for bending moment equal to

$$M_{emp}^+ = 6.09 \left(\frac{10.93}{8.00} \right) \left(\frac{1}{1.20} \right) \left(\frac{1}{1.33} \right) = 5.21 \text{ k} - \text{ft} = 7.1 \text{ kN} - \text{m}$$

As expected, the empirical design procedure provides a conservative estimate for the interior bending moments within the bridge deck. The process also provides a conservative estimate for the bending moments in the exterior span. It is also interesting to note that the present wheel loading is 37% greater than the HL-93 truck wheel loading. As a result, the bending moments used at near service-level evaluations of bridge deck response are conservative when compared to the in-situ magnitudes estimated using the methodology developed in the present study.

CONCLUSIONS

An in-situ load test performed on an FRP-SIP bridge deck system was discussed. Measurement of strains on the FRP-SIP formwork using portable strain sensors developed specifically for a WisDOT bridge monitoring research effort was also evaluated. Description and application of a simple methodology to analyze the equivalent bending moment in the deck and the corresponding wheel load distribution width within the deck slab was also included. It was found that the Standard (AASHTO 2002) and LRFD procedures (AASHTO 2006) for estimating bending

moment load-transfer mechanisms in bridges at load levels slightly larger than service-level loads yield conservative predictions for the load distribution width for this FRP-SIP deck system.

The results reported in the present manuscript are the first of a series of two load tests designed to monitor the behavior of this novel bridge deck system over time. The Marquette University research team is currently using the results of the field load testing to evaluate finite element models of the bridge deck system and these models will help to further clarify and understand the data measured in the field. These models can also be used to develop refined estimates for bending moment strip width expressions for this type of bridge deck system.

ACKNOWLEDGEMENTS

This research was funded by the Wisconsin Department of Transportation (Contract: 0092-05-02) and the U.S. Department of Transportation in the interest of information exchange. The material or information reported is the result of research done under the auspices of the Department and the Wisconsin Highway Research Program. The content reflects the views of the authors. WisDOT and FHWA (US DOT) assumes no liability for its contents or use thereof. This publication does not constitute a standard specification or regulation of the Department or FHWA.

REFERENCES

- AASHTO. (2002). *Standard Specifications for Highway Bridges, Customary Units, 17th Edition*, American Association of State Highway and Transportation Officials, Washington, DC.
- AASHTO. (2006). *AASHTO LRFD Bridge Design Specifications Including 2006 Interim Revisions, Customary U.S. Units, 3rd Edition*, American Association of State Highway and Transportation Officials, Washington, DC.
- Allen, J. H. (1991). "Cracking, Serviceability, and Strength of Concrete Bridge Decks." *Transportation Research Record*, No. 1290, 152-171.
- Bank, L. C., Oliva, M. G., and Russell, J. S. (2005). "In-Situ Load Testing of IBRC Bridge." Wisconsin Department of Transportation, Federal Highway Administration, Madison, WI.
- Batchelor, B., Hewitt, B. E., Csagoly, P., and Holowka, M. (1978). "Investigation of the Ultimate Strength of Deck Slabs of Composite Steel/Concrete Bridges." *Transportation Research Record*, No. 664, 162-170.
- Beal, D. B. (1982). "Load Capacity of Concrete Bridge Decks." *Journal of the Structural Division*, 108(ST4), 814-832.
- Berg, A. C. (2004). "Analysis of a Bridge Deck Built on U.S. Highway 151 with FRP Stay-In-Place Forms, FRP Grids, and FRP Rebars," MS Thesis, University of Wisconsin at Madison, Madison, WI.
- Berg, A. C., Bank, L. C., Oliva, M. G., Russell, J. S., and Jeffrey, S. (2004). "Construction of a FRP Reinforced Bridge Deck on U.S. Highway 151 in Wisconsin." 83rd Annual Meeting of the Transportation Research Board, National Research Council, Transportation Research Board, Washington, DC, CD-ROM.
- Cao, L. C., Allen, J. H., Shing, P. B., and Woodham, D. (1996). "Behavior of RC Bridge Decks with Flexible Girders." *Journal of Structural Engineering*, 122(1), 11-19.
- Cao, L. C., and Shing, P. B. (1999). "Simplified Analysis Method for Slab-On-Girder Highway Bridges." *Journal of Structural Engineering*, 125(1), 49-59.
- Csagoly, P., Holowka, M., and Dorton, R. (1978). "The True Behavior of Thin Concrete Bridge Slabs." *Transportation Research Record*, No. 664, 171-179.
- Dieter, D. A. (2002). "Experimental and Analytical Study of Concrete Bridge Decks Constructed with FRP Stay-In-Place Forms and Grid Reinforcing," MS Thesis, University of Wisconsin at Madison, Madison, WI.
- Dieter, D. A., Dietsche, J. A., Bank, L. C., Oliva, M. G., and Russell, J. S. (2002a). "Concrete Bridge Decks Constructed with Fiber-Reinforced Polymer Stay-In-Place Forms and Grid Reinforcing." Annual Meeting of the Transportation Research Board, National Research Council, Transportation Research Board, Washington, D.C., CD-ROM.
- Dieter, D. A., Dietsche, J. S., Bank, L. C., Oliva, M. G., Russell, J. S., and Jeffrey, S. (2002b). "Concrete Bridge Decks Constructed with Fiber-Reinforced Polymer Stay-In-Place Forms and Grid Reinforcing." 81st Annual Meeting of the Transportation Research Board, National Research Council, Transportation Research Board, Washington, DC, CD-ROM.
- Dietsche, J. S. (2002). "Development of Material Specifications for FRP Structural Elements for the Reinforcing of a Concrete Bridge Deck," MS Thesis, University of Wisconsin at Madison, Madison, WI.
- Fang, I. K., Worley, J., Burns, N. H., and Klingner, R. E. (1990). "Behavior of Isotropic R/C Bridge Decks on Steel Girders." *Journal of the Structural Division*, 116(3), 659-678.

- Helm Mueller, E. J., Bank, L. C., Dieter, D. A., Dietsche, J. A., Oliva, M. G., and Russell, J. S. (2002). "The Effect of Freeze-Thaw on Bond Between FRP Stay-In-Place Deck Forms and Concrete." CDCC 2002, 2nd International Conference on Durability of Fiber Reinforced Polymer (FRP) Composites for Construction, Montreal, Quebec, CAN, 1643-1654.
- Martin, K. E. (2006). "Impact of Environmental Effects on, and Condition Assessment of IBRC Bridge Decks in Wisconsin," M.S. Thesis, Marquette University, Milwaukee, WI.
- Newmark, N. M. (1949). "Design of I-Beam Bridges." *Transactions of the ASCE*, 114, 997-1022.
- Ringelstetter, T. E., Bank, L. C., Oliva, M. G., Russell, J. S., Matta, F., and Nanni, A. (2006). "Development of a Cost-Effective Structural FRP Stay-In-Place Formwork System for Accelerated and Durable Bridge Deck Construction." Annual Meeting of the Transportation Research Board, National Research Council, Transportation Research Board, Washington, DC.
- Schneeman. (2006). "Development and Evaluation of a Removable and Portable Strain Sensor for Short-Term Live Loading of Bridge Structures," M.S. Thesis, Marquette University, Milwaukee, WI.
- Wan, B., Foley, C. M., and Martin, K. E. "Freeze-Thaw Cycling Effects on Shear Transfer Between FRP Stay-in-Place Formwork and Concrete." *The Third International Conference on Durability and Field Applications of Fiber Reinforced Polymer (FRP) Composites for Construction, CDCC 2007*, Quebec City, CAN, 227-234.
- Westergaard, H. M. (1930). "Computation of Stresses in Bridge Slabs Due to Wheel Loads." *Public Roads*, March, 1-23.

Table 1 – Strain Readings Recorded During Field Loading Tests. (Quantities shown in bold-face font are field-measured quantities.)

Span and Stop Location	Location Relative to Truck Front Wheel								
	Strain ($\mu\epsilon$)								
	-70 in. -1.8 m	-52.5 in. -1.3 m	-35 in. -890 mm	-17.5 in. -440 mm	Front Wheel	17.5 in. 440 mm	35 in. 890 mm	52.5 in. 1.3 m	70 in. 1.8 m
Exterior Span									
TW1-W2	9	26	23	32	92	32	23	26	9
TW1-W1	n.a.	14	41	59	45	41	41	14	n.a.
TW1-M	n.a.	n.a.	34	26	76	69	21	n.a.	n.a.
TW1-E1	n.a.	22	12	50	126	40	12	22	n.a.
TW1-E2	16	5	30	75	75	75	30	5	16
Interior Span									
TW2-W2	2	6	15	48	51	48	15	6	2
TW2-W1	n.a.	4	11	32	76	24	11	4	n.a.
TW2-M	n.a.	n.a.	18	47	43	20	10	n.a.	n.a.
TW2-E1	n.a.	13	28	25	35	21	25	28	n.a.
TW2-E2	9	19	16	22	36	22	16	19	9

Table 2 – Averaged Bending Moment Computed at Each Strain Gauge Location.

Span and Stop Location	Location Relative to Truck Front Wheel								
	Average Bending Moment $\left[\begin{matrix} \text{k-ft} \\ (\text{kN-m}) \end{matrix} \right]$								
	-70 in. -1.8 m	-52.5 in. -1.3 m	-35 in. -890 mm	-17.5 in. -440 mm	Front Wheel	17.5 in. 440 mm	35 in. 890 mm	52.5 in. 1.3 m	70 in 1.8 m
Exterior Span									
TW1-W2	0.50 (0.68)	1.44 (2.0)	1.27 (1.7)	1.77 (2.4)	5.09 (6.9)	1.77 (2.4)	1.27 (1.7)	1.44 (2.0)	0.50 (0.7)
TW1-W1	n.a.	0.77 (1.0)	2.27 (3.1)	3.26 (4.4)	2.49 (3.4)	2.27 (3.1)	2.27 (3.1)	0.77 (1.0)	n.a.
TW1-M	n.a.	n.a.	1.88 (2.55)	1.44 (2.0)	4.21 (5.7)	3.82 (5.2)	1.16 (1.6)	n.a.	n.a.
TW1-E1	n.a.	1.22 (1.7)	0.66 (0.89)	2.77 (3.8)	6.97 (9.4)	2.21 (3.0)	0.66 (0.89)	1.22 (1.7)	n.a.
TW1-E2	0.89 (1.2)	0.28 (0.38)	1.66 (2.2)	4.15 (5.6)	4.15 (5.6)	4.15 (5.6)	1.66 (2.2)	0.28 (0.38)	0.89 (1.2)
Interior Span									
TW2-W2	0.11 (0.15)	0.33 (0.45)	0.83 (1.1)	2.66 (3.6)	2.82 (3.8)	2.66 (3.6)	0.83 (1.1)	0.33 (0.45)	0.11 (0.15)
TW2-W1	n.a.	0.22 (0.30)	0.61 (0.83)	1.77 (2.4)	4.21 (5.7)	1.33 (1.8)	0.61 (0.83)	0.22 (0.30)	n.a.
TW2-M	n.a.	n.a.	1.00 (1.4)	2.60 (3.5)	2.38 (3.2)	1.11 (1.5)	0.55 (0.75)	n.a.	n.a.
TW2-E1	n.a.	0.72 (0.98)	1.55 (2.1)	1.38 (1.9)	1.94 (2.6)	1.16 (1.6)	1.38 (1.9)	1.55 (2.1)	n.a.
TW2-E2	0.50 (0.68)	1.05 (1.4)	0.89 (1.2)	1.22 (1.6)	1.99 (2.7)	1.22 (1.6)	0.89 (1.2)	1.05 (1.4)	0.50 (0.68)

Table 3 – Bending Moment (per foot) and Strip Width Comparison.

Span and Stop Location	M_T^+ [k-ft (kN-m)]	M_{avg}^+ [k-ft/ft (kN-m/m)]	M_{STD} [k-ft/ft (kN-m/m)]	SW^+ [in. (m)]	SW_{LRFD}^- [in. (m)]	SW_{LRFD}^+ [in. (m)]
Exterior Span						
TW1-W2	15.05 (20.4)	1.15 (5.1)	2.92 (13.0)	155.4 (3.9)	74.08 (1.9)	83.38 (2.1)
TW1-W1	14.11 (19.1)	1.38 (6.1)		128.9 (3.3)		
TW1-M	12.51 (17.0)	1.72 (7.7)		103.9 (2.6)		
TW1-E1	15.72 (21.3)	1.54 (6.8)		115.7 (2.9)		
TW1-E2	18.09 (24.5)	1.38 (6.1)		129.2 (3.3)		
Interior Span						
TW2-W2	10.68 (14.5)	0.81 (3.6)	2.92 (13.0)	175.2 (4.5)	74.08 (1.9)	83.38 (2.1)
TW2-W1	8.96 (12.1)	0.88 (3.9)		162.3 (4.1)		
TW2-M	7.64 (10.4)	1.05 (4.7)		136.1 (3.5)		
TW2-E1	9.68 (13.1)	0.95 (4.2)		150.3 (3.8)		
TW2-E2	9.30 (12.6)	0.71 (3.2)		201.2 (5.1)		

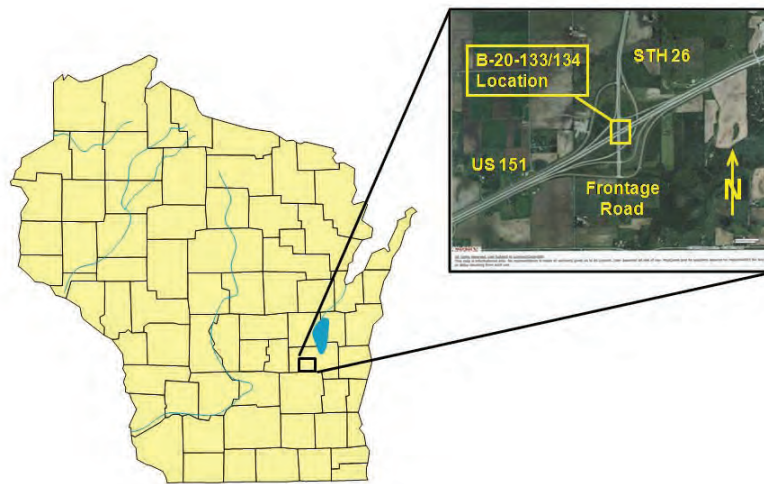


Figure 1 – Bridge B-20-133 Located in Waupun, Wisconsin.

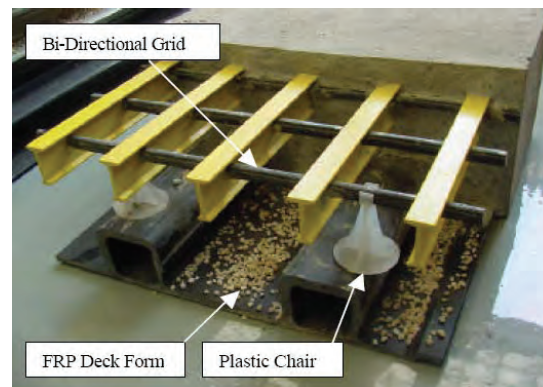


Figure 2 – Stay-In-Place Bridge Deck System Mockup (Dieter et al. 2002a).



AVA Documents
Document Sharing Hub

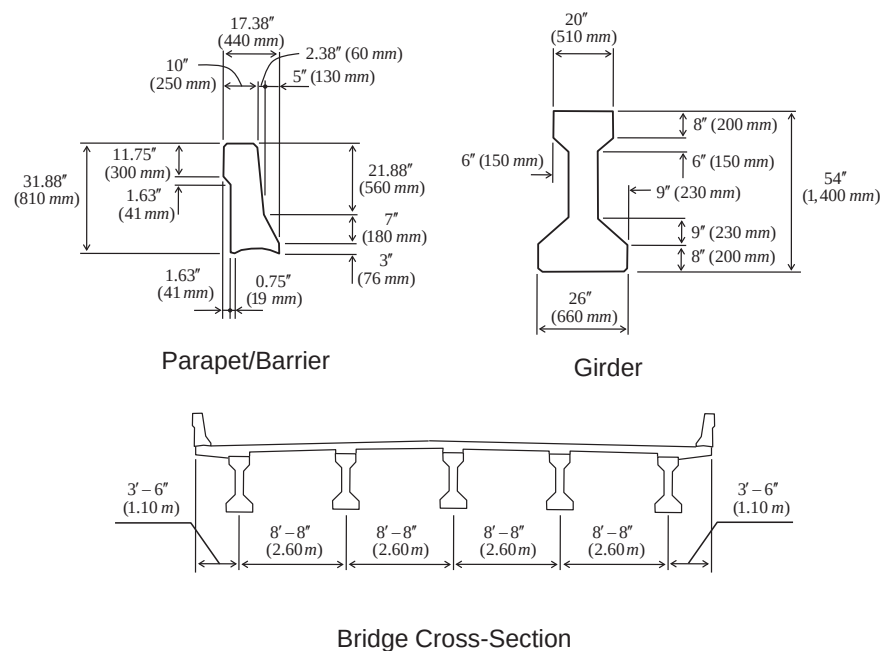
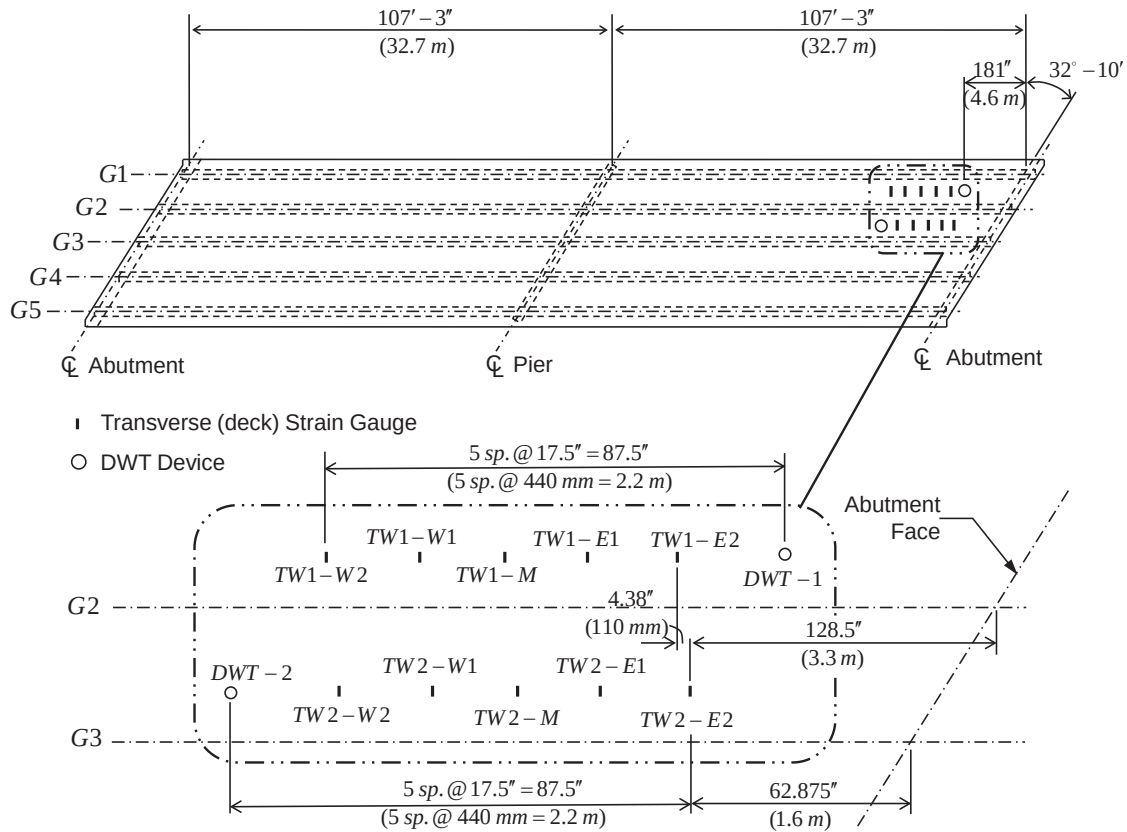


Figure 5 – Bridge B-20-133 Cross-Section with Girder and Parapet/Barrier Dimensions.

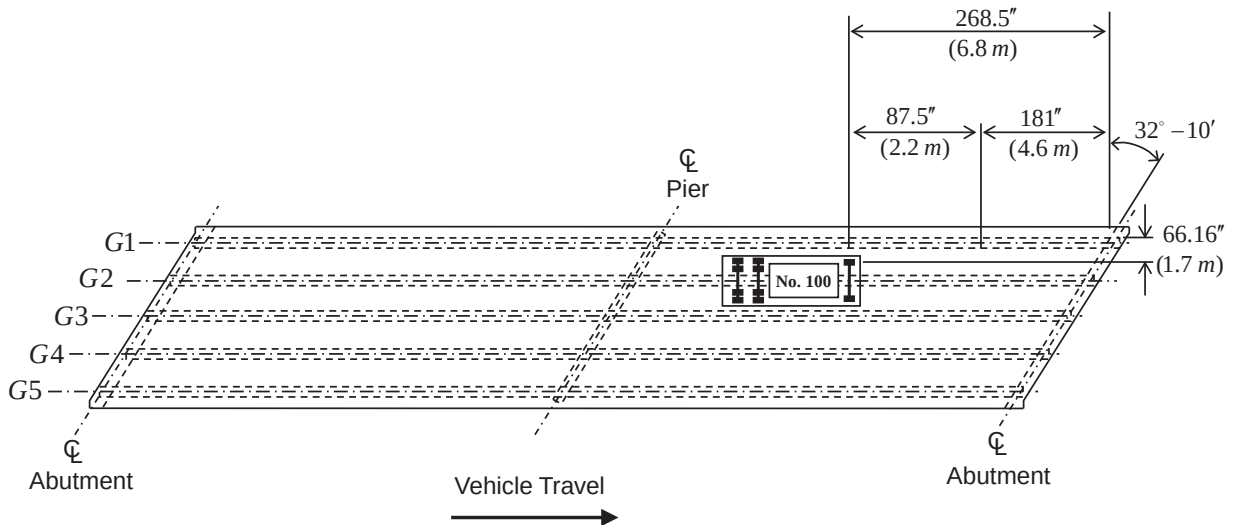


Figure 6 – Loading Protocol 1: Positive Bending Moment Strains in Exterior Span.

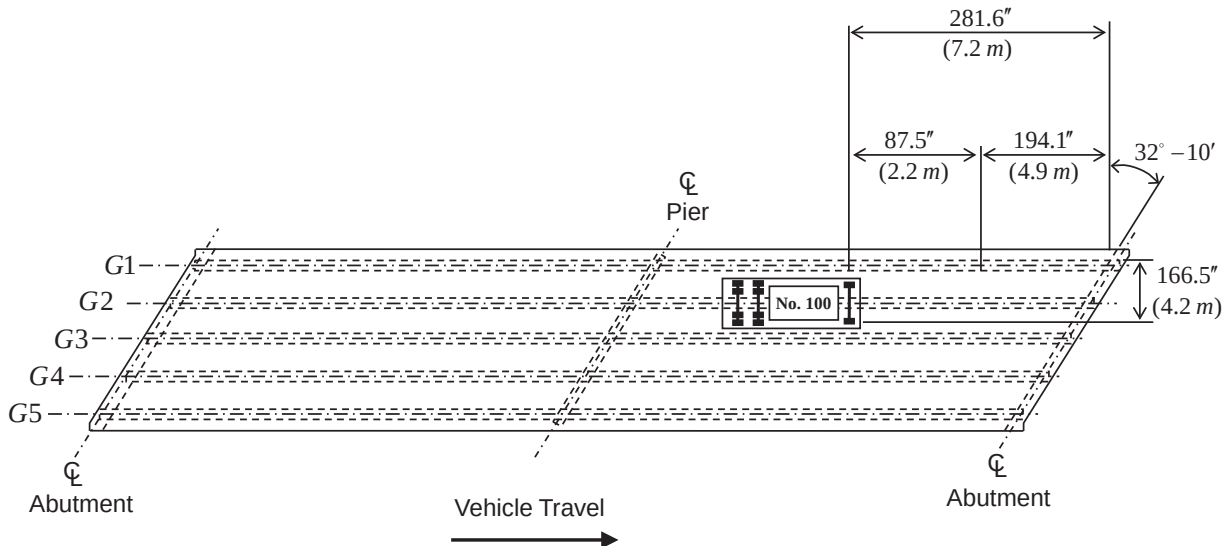


Figure 7 – Loading Protocol 2: Positive Bending Moment Strains in Interior Span.



(a) Truck Approaching Deck Targets



(b) Truck Wheel Positioned Over Deck Target (Strain Gauge in this Case)



(c) Truck Positioned at Last Deck Target in Loading Protocol with Pause

Figure 8 – Truck Positioning for Bridge Deck Loading.

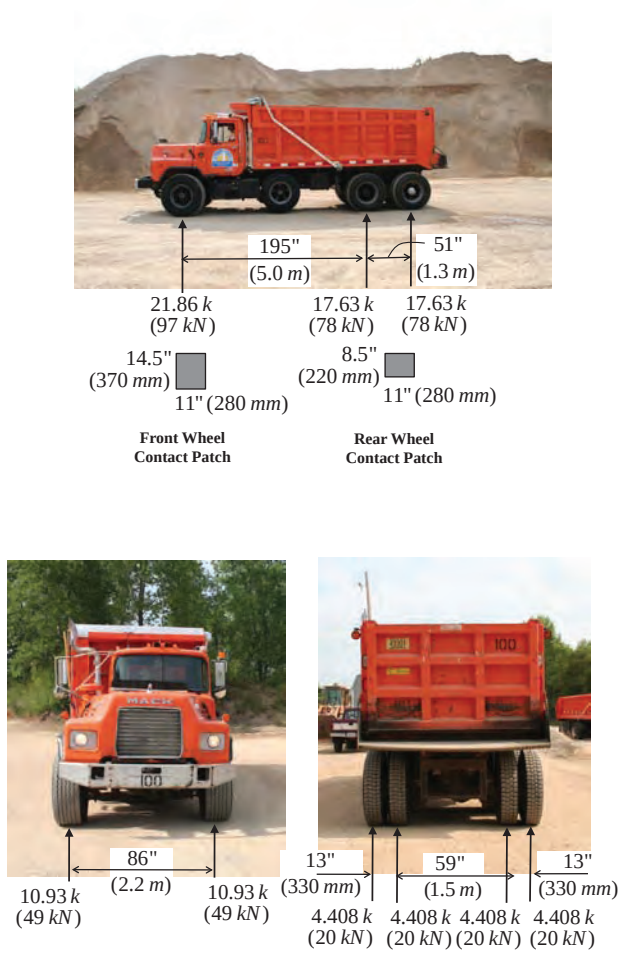


Figure 9 – Calibrated Tri-Axle Dump Truck with Axle Configuration used in the Testing.

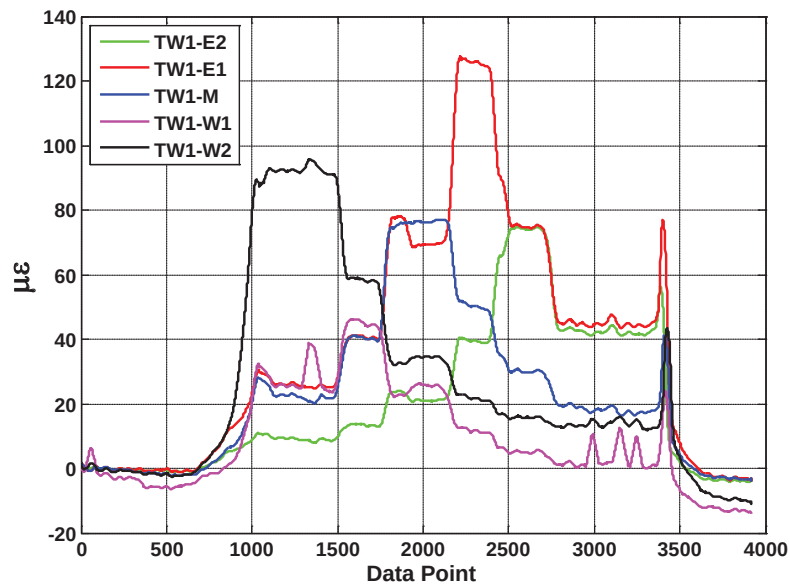


Figure 10 – Strain Data for TW1 Series Sensors (Exterior Bridge Deck Span).

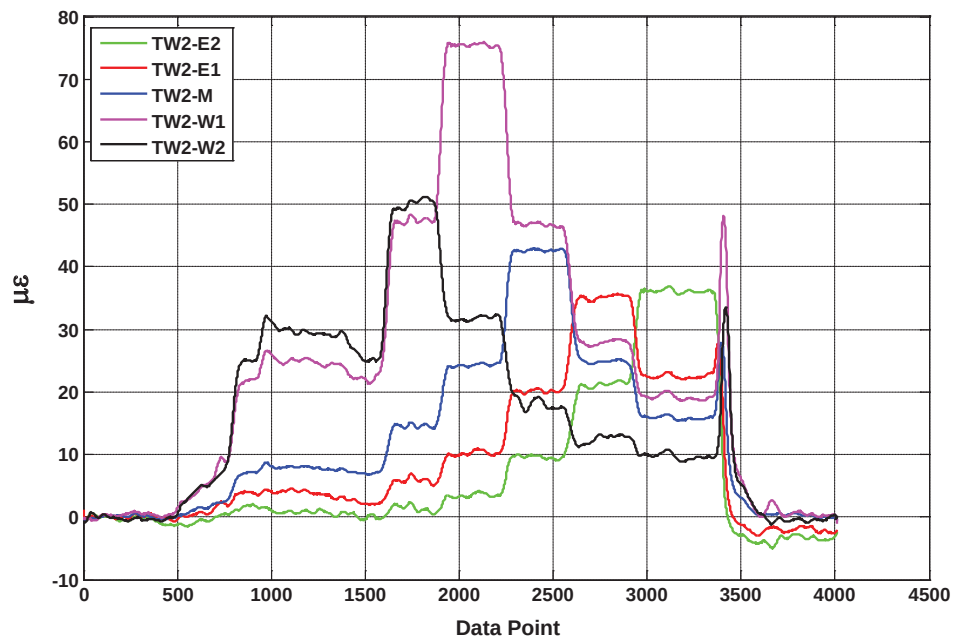


Figure 11 – Strain Data for TW2 Series Sensors (Interior Bridge Deck Span).