Comparison of the Fatigue Behaviors of FRP Bridge Decks and Reinforced Concrete Conventional Decks Under Extreme Environmental Conditions

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This paper summarizes the results of the fatigue test of four composite bridge decks in extreme temperatures (−30℃ and 50℃). The work was performed as part of a research program to evaluate and install multiple FRP bridge deck systems in Dayton, Ohio. A two-span continuous concrete deck was also built on three steel girders for the benchmark tests. Simulated wheel loads were applied simultaneously at two points by two servo-controlled hydraulic actuators specially designed and fabricated to perform under extreme temperatures. Each deck was initially subjected to one million wheel load cycles at low temperature and another one million cycles at high temperature. The results presented in this paper correspond to the fatigue response of each deck for four million load cycles at low temperature and another four million cycles at high temperature. Thus, the deck was subjected to a total of ten million cycles. Quasi-static load-deflection and load-strain responses were determined at predetermined fatigue cycle levels. Except for the progressive reduction in stiffness, no significant distress was observed in any of the composite deck prototypes during ten million load cycles. The effects of extreme temperatures and accumulated load cycles on the load-deflection and load-strain response of FRP composite and FRP-concrete hybrid bridge decks are discussed based on the experimental results.

Key Words : Composites Bridge Deck, Fatigue, Quasi-static Test, Pultrusion, Vacuum Assistant Resin Transfer Molding (VARTM), Impact Loading Cycle, Contact Molding ; Hand Lay-Up, Extreme Environment, Cold Regions

1. Introduction

Composite bridge decks are becoming an attractive solution in mitigating or minimizing aging or corrosion-related degradation or seismic damages to highway bridges because of their excellent durability and higher energy absorption and dissipation characteristics compared to concrete decks. However, construction of highway bridges with FRP deck materials requires an understanding of their life-cycle estimate and performance under traffic loads. Traffic loads, mainly due to heavy trucks, induce repetitive stress cycles on bridge decks during the service life of the structure. Past research has shown that bridge decks made of pultruded FRP composites have extended fatigue life at ambient temperature

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(Kim and Kim, 1993 ; Bakeri and Sunder, 1997 ; Lopez-Anido et al., 1998 ; Lopez-Anido et al., 1999 ; Kim et al., 2000 ; Chung et al., 2001 ; Kwon et al., 2001 ; Kim and Kim, 2001 ; Dutta et al., 2002a ; Dutta et al., 2002b). However, the synergistic effect of high-temperature and fatigue cycling requires further studies.

The FRP deck system was planned to be used on two side-by-side five-span continuous bridges on route 49 in Dayton, Ohio. The long spans of the bridges would allow the evaluation of FRP deck systems from four manufacturers under similar loading and environmental conditions. The complete test program included the static and failure evaluation of FRP deck panels, the accelerated aging of FRP deck material that is representative of harsh environmental conditions, and the fatigue testing of FRP deck-steel beam systems under extreme temperatures. The static and failure evaluation of FRP-concrete decks was performed at the University of Kentucky (Lopez-Anido et al., 1999). The accelerated environmental durability evaluation was conducted at the University of Maine (Lopez-Anido et al., 2001) and the fatigue evaluation was conducted at the U.S. Army ERDC-CRREL (Engineer Research and Development Center-Cold Regions Research and Engineering Laboratory,

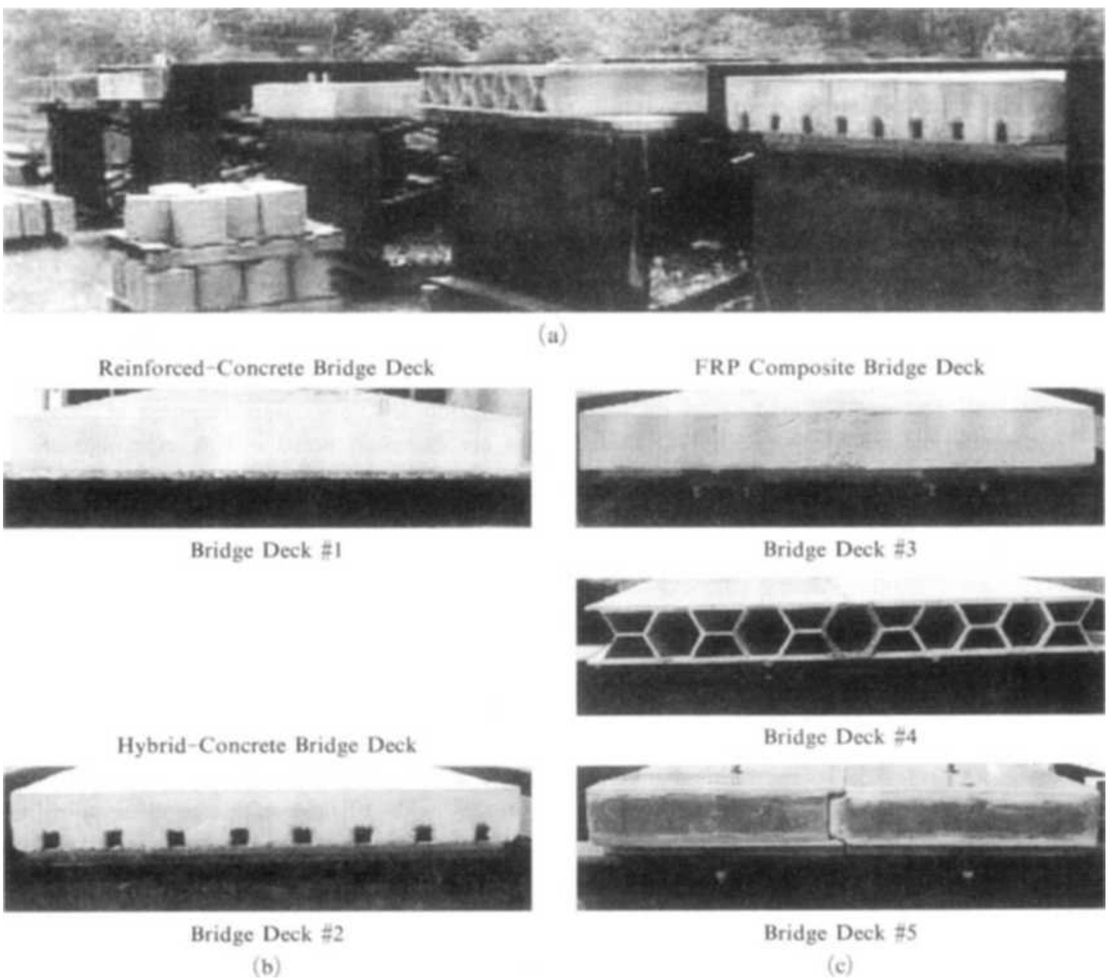

Fig. 1 FRP deck prototypes (a) Entire view, (b) Front views of reinforced-concrete bridge decks, and (c) Front view of FRP composite bridge deck

Hanover, NH, U.S.A.).

In this study, the experimental fatigue evaluation of five deck prototypes, which included three full-size FRP composite bridge decks, one hybrid FRP-concrete deck, and one reinforced-concrete conventional bridge deck, was conducted (Fig. 1). The deck prototypes were evaluated under two extreme temperatures to assess the fatigue-temperature response. Each deck prototype was initially subjected to one million simulated wheel load cycles at the low temperature of −30℃, and another one million cycles at the controlled high temperature of 50℃. The Initial test results of the current series of tests were reported recently (Lopez-Anido et al., 2001). The results presented in this paper correspond to testing each deck prototype for an additional four million cycles at the low temperature and four million cycles at the high temperature. Quasi-static load tests were conducted at specific intervals during fatigue cycling to evaluate the load-deflection and load-strain responses at several deck locations. The experimental results were correlated with the performance of a conventional reinforced-concrete deck subjected to the same series of tests.

2. Test Materials

2.1 Conventional reinforced-concrete deck

The conventional reinforced-concrete deck (Bridge #1, Fig. 1(b) top) was designed for the benchmark response of the set of FRP bridge decks. This deck had dimensions of 1.828×6.100×0.178 m and was connected to the supporting W 36×182 steel girders using shear studs. In the direction perpendicular to the girders, top steel reinforcing bars were placed in the concrete slab with a spacing of 133 mm and a cover of 38.1 mm. A bottom reinforcement layer was placed with a spacing of 152.4 mm.

2.2 Hybrid FRP-concrete deck

The hybrid FRP-concrete deck system (Bridge #2, Fig. 1(b) bottom) had FRP pultruded panels that were used for stay-in-place formwork and concrete reinforcement. The pultruded panels had a width of 457 mm and two stiffening tubular cells with a height of 76 mm. This FRP material was reinforced with E-glass roving and directional-bias fabric in a polyester-vinyl ester resin blend. Concrete was cast on the FRP panels to attain the specified slab depth of 203 mm. Top reinforcement in both directions was provided by non-corrosive E-glass rebar with deformations to improve the bond with the concrete. The deck was connected to the supporting steel girders using shear studs. After placing the pultruded panels on the steel girders, shear studs were welded. This test specimen had dimensions of 1.828×6.100×0.203 m. A concrete haunch was placed between the FRP deck panels and the steel girders.

2.3 FRP deck fabricated by the VARTM process

Test deck #3 (Fig. 1(c) top) was fabricated by the Vacuum-Assisted Resin Transfer Molding (VARTM) process. The deck was composed of vinyl ester resin (Dow Derakane 411), a multi-axial stitched E-glass fabric (0/90/ρ45) (BTI, QM6408), and an integral vertical foam-filled cell core that was wrapped with an E-glass composite. This deck prototype had dimensions of 1.828×6.100×0.203 m. Two panels of 0.914 m in width and 6.100 m in length were connected with a longitudinal joint (perpendicular to the girder direction) to form the deck prototype while providing a smooth surface.

2.4 Pultruded FRP deck

The test deck system #4 (Fig. 1(c) middle) was made of pultruded FRP profiles that were placed transversely to the girder direction. The section was composed of hexagonal and double trapezoidal profiles, which were bonded together with a high-strength adhesive to form prefabricated panels. The design involved pultruded interlocking FRP sections that acted as beams to transfer the loads to the girder. This deck prototype had dimensions of 1.828×6.100×0.203 m. The material constituents were vinyl ester resin (Reichhold, Atlac 580-05), E-glass continuous roving, and a multi-axial stitched E-glass fabric (90/ρ45) (BTI, TH4000/THX1501).

2.5 FRP Deck fabricated by the contact molding hand lay-up process

Test deck system #5 (Fig. 1(c) bottom) consisted of a honeycomb core sandwiched between two contact-molded hand lay-up face plates. The matrix was made of isophthalic/terephthalic polymer resin (AOC, Vibrin F457-BRP-25), which was reinforced with bi-axial E-glass fabric (0/90) and mat (BTI, CM4810 & CCC A118). The dimensions of the deck prototype were 1.828×6.100×0.203 m.

3. Experimental Setup

3.1 Self-reacting loading frame

Each deck prototype was placed on three W36×182 steel girders, resulting in a continuous two-span bridge structure [Fig. 2(a)]. A self-reacting steel test frame was designed for a maximum load capacity of 270 kN. The maximum deflection of the steel transverse beam was limited to less than 0.25 mm. Two actuators mounted on the two cross arms of this load frame applied the load through two steel plates of 228×559 mm centered with respect to the supports of the span, which simulate the AASHTO HS20-44 design truck wheel load print. The long dimension of the plate was perpendicular to the girder direction.

The inner long edge of the plate was 178 mm away from the center of the deck. An elastomeric pad was placed between each steel plate and the prototypes to provide uniform pressure that simulates the wheel load action [Fig. 2(b)]. The setup induced a positive bending moment under the load and a negative bending moment on the central support.

3.2 Instrumentation

Each test deck was instrumented with strain gages (EA-5-500BL-350, Micro-Measurements),

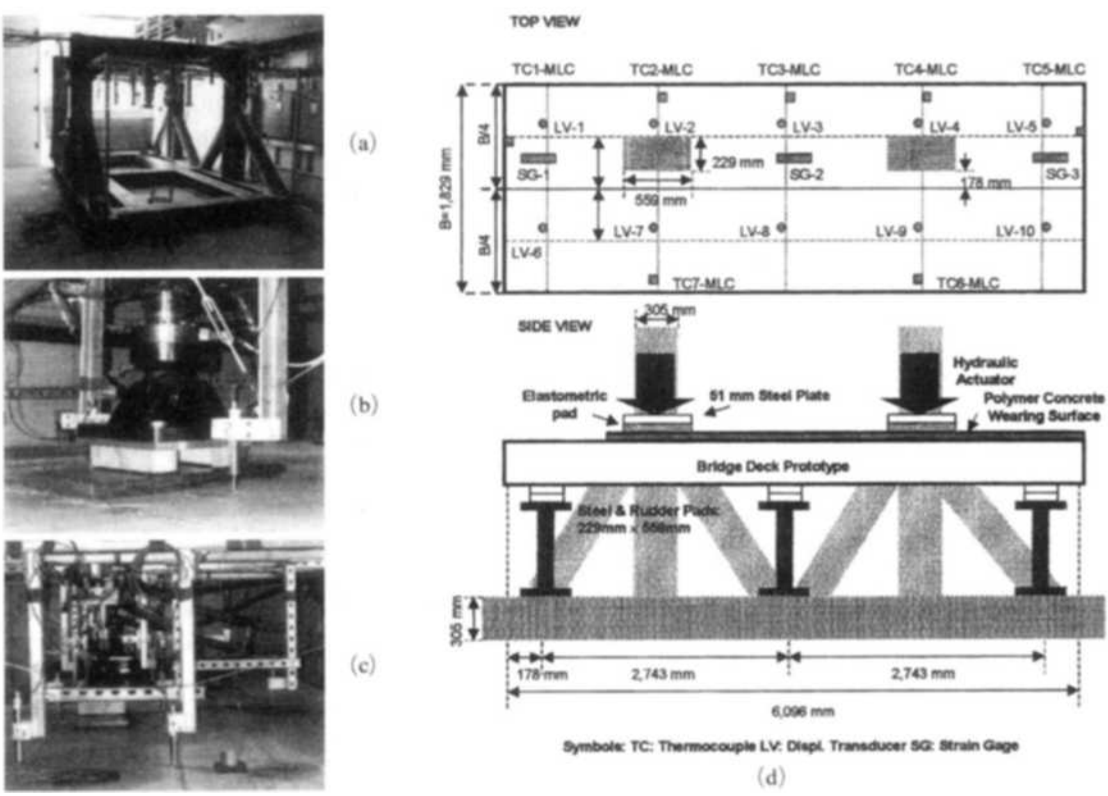

Fig. 2 Experimental setup: (a) Self-reacting test frame for fatigue loading, (b) Hydraulic actuator, (c) Strain gages and LVDTs and (d) Test setup and instrumentation layout

thermocouples, and linear voltage differential transducers (LVDTs), which were supported by an independent steel frame. The LVDTs were used to measure deflections on the top and bottom surfaces of the decks. Seven thermocouples were used on the four sides of each deck, and one thermocouple was used for ambient temperature [Fig. 2(c)]. The locations of the strain gages bonded to the bottom deck surface were symmetrical with respect to the numerically matching strain gages bonded to the top surface of decks. The complete instrumentation layout is shown in Fig. 2(d).

3.3 Fatigue test procedure

The fatigue evaluation procedure consisted of applying four million simulated wheel load cycles at −30℃ and another four million cycles at 50℃. The fatigue performance of each FRP deck prototype was compared with the response of the conventional reinforced-concrete deck. The fatigue load range was computed for an AASHTO HS20-44 truck wheel with impact and maximum load. A computed load was applied simultaneously at two points by servo-controlled hydraulic actuators specially designed and fabricated for this study. The maximum applied load (P_{max}) was 115 kN. The minimum applied load (P_{min}) was 9 kN. Therefore, the fatigue stress ration ($R=P_{min}/P_{max}$) was 0.077. Loading was applied using a sinusoidal waveform with a frequency of 3.5 Hz.

Quasi-static load deflection tests were performed at regular intervals. In the quasi-static test the load was applied at a rate of 1 mm/min. and sensor measurements were recorded every 3 seconds. Each quasi-static test consisted of a loading and unloading cycle and was repeated three times, as shown in Fig. 3.

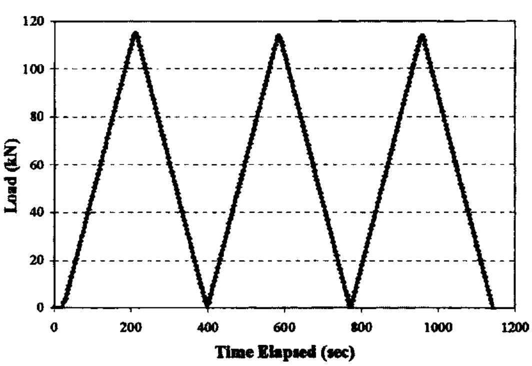

Fig. 3 Typical quasi-static load test

4. Discussions

The fatigue deck prototypes were not anticipated to fail during the loading cycles. However, following the ten million cycles of loading at two extreme temperatures, degradation of stiffness was expected (Figs. 4~5). Fatigue damage accumulation can induce the stiffness degradation of the FRP composite deck material.

Fatigue damage can also lead to residual deformation in the deck and in the deck-girder haunch connections. Thus, the fatigue performance evaluation was based on assessing the residual stiffness of the deck response and the fatigue damage. Quasi-static load deflection tests were conducted for damage assessment. The experimental data were analyzed and load-deflection curves were generated.

The load-deflection curves at the low temperature, of −30℃, and the high temperature of 50℃, at the five LVDT locations on the top of each panel and aligned in the direction perpendicular to girders are shown in Fig. 4. The reinforced-concrete deck (Bridge #1) and the FRP-concrete hybrid deck (Bridge #2) exhibited higher stiffness than the FRP composite decks (Bridges #3, #4, and #5).

Load deflection curves for each deck prototype for the LVDT position LV2 before fatigue cycling, after 2 million load cycles and after 10 million load cycles are shown in Fig. 5. The decrease in slope of the load-deflection curves with number of fatigue cycles, indicates damage accumulation in the decks.

The effects of temperature on the load-deflection response are presented in Fig. 6. As expected, the deck stiffness was reduced at the higher temperature level. The reduction in stiffness with temperature was more important for the FRP composite decks than for the reinforced-concrete deck and the FRP-concrete deck. From the maximum load-deflection curves (Fig. 6), it was observed that the FRP bridge deck fabricated by the VARTM process (Bridge #3) and the FRP bridge deck fabricated by the pultrusion process (Bridge #4) had significantly more deflection than that of reinforced-concrete deck (Bridge #1)

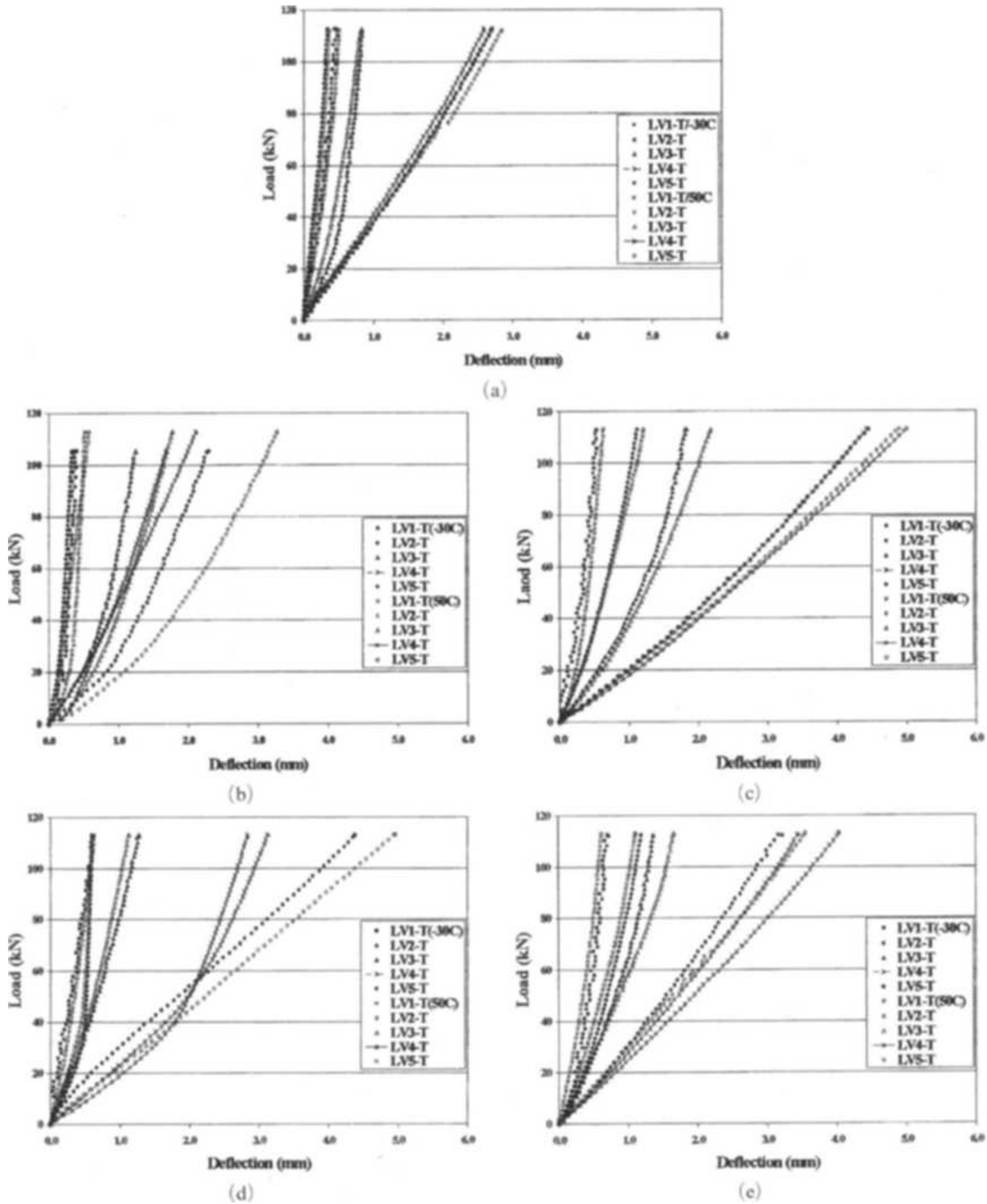

Fig. 4 Load-deflection curves after ten million cycles of fatigue loading: (a) Bridge #1, (b) Bridge #2, (c) Bridge #3, (d) Bridge #4 and (e) Bridge #5

and the FRP-composite hybrid deck (Bridge #2). There was only a relatively small change in deck stiffness between the FRP bridge deck fabricated by the hand lay-up contact molding process (Bridge #5) and the reinforced-concrete bridge deck (Bridge #1) at both low and high temperatures.

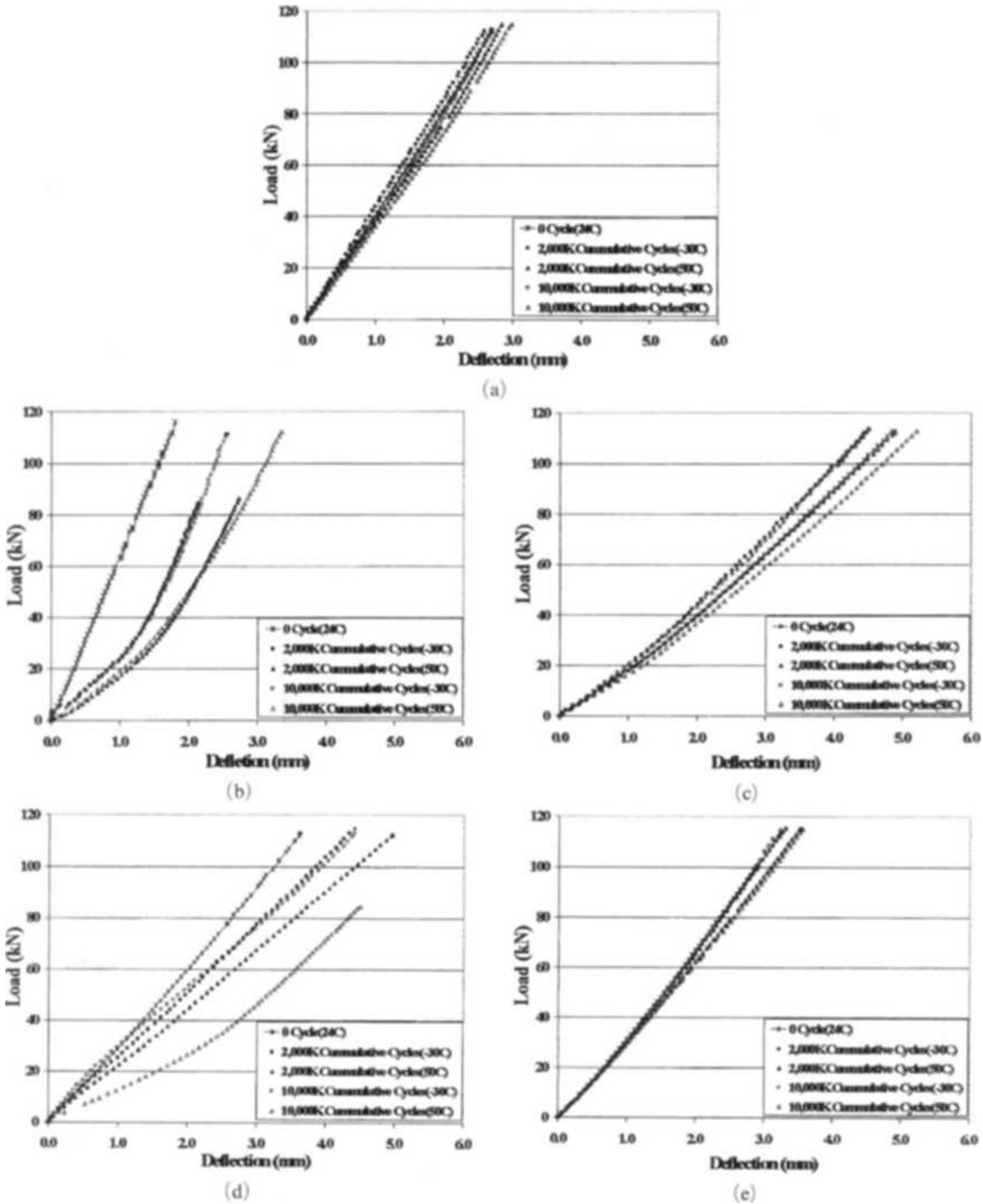

Fig. 5 Load-deflection curves at location LV-2 at different load cycles intervals: (a) Bridge #1, (b) Bridge #2, (c) Bridge #3, (d) Bridge #4 and (e) Bridge #5

Low-temperature and high-temperature load-strain curves were obtained from the strain gage SG2 measurements, as shown in Fig. 7. The curves for the deck prototype at each temperature level indicate that there was a significant difference in strain between the FRP composite decks and the reinforced-concrete deck. However, there was almost no difference in the load-strain response between the FRP composite bridge deck fabricated by the pultrusion process (Bridge #4) and the FRP bridge deck fabricated by the VARTM process (Bridge #3). The hybrid FRP-concrete bridge deck (Bridge #2) was stiffer (higher load-strain slope) than the reinforced-concrete deck (Bridge #1), as shown in Fig. 7. This difference is attributed to the greater thickness of the hybrid FRP-concrete deck compared to the reinforced-concrete deck.

The significant stiffness change with temperature implies that the deck stiffness is controlled mainly by temperature changes and not by the number of applied load cycles. All the curves

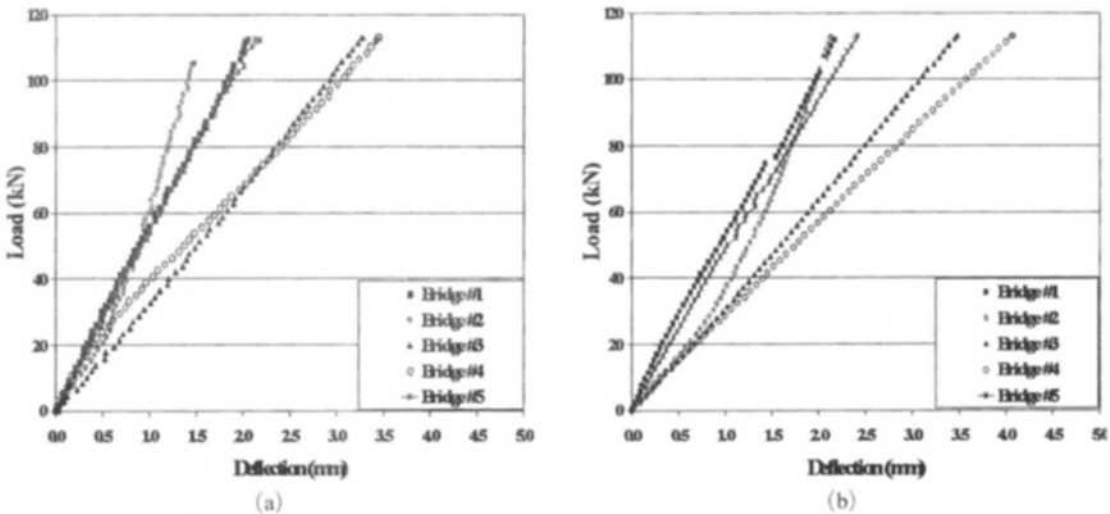

Fig. 6 Effects of Temperature on the Stiffness of Deck Prototypes: (a) Low Temperature and (b) High Temperature

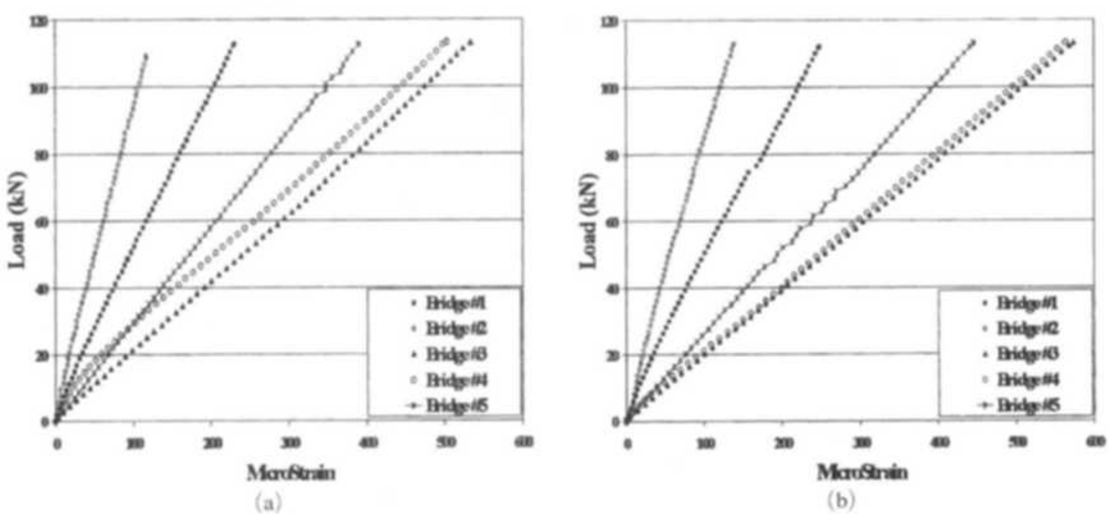

Fig. 7 Load-strain curves for deck prototypes: (a) Low temperature and (b) High temperature

exhibited a load-deflection relationship that was approximately linear within the loading range, indicating that there was no slip in the longitudinal joint.

The deck prototypes were inspected visually for signs of distress, such as cracks and damage at connections after fatigue cycling. No damage was visible in the three FRP composite decks (Bridge #3, Bridge #4, and Bridge #5), however hairline cracks were observed in the tension region over the FRP-concrete hybrid deck (Bridge #2).

5. Conclusions

The general conclusions drawn from the results of the investigation are:

(1) FRP composite deck prototypes demonstrated satisfactory performance in highway bridge applications where extreme temperature conditions are expected.

(2) The stiffness of the FRP composite decks, measured from the load-deflection curves, was substantially affected by extreme temperature levels.

(3) Progressive degradation in stiffness with load cycling was observed for all deck prototypes under the two extreme temperatures of −30℃ and 50℃.

(4) The stiffness of FRP composite decks under simulated wheel loads was more susceptible to the selected extreme temperature changes than to ten million cumulative load cycles.

General Remarks

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