DESIGN, CONSTRUCTION AND MONITORING OF FOUR INNOVATIVE CONCRETE BRIDGE DECKS USING NON-CORROSIVE FRP COMPOSITE BARS

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ABSTRACT

This paper presents the construction details, testing, and monitoring results of four FRP reinforced concrete bridges recently constructed in North America. Three bridges, Joffre Bridge, Wotton Bridge, and Magog Bridge, are located in Quebec, Canada. While the fourth one, Morristown Bridge, is located in Vermont, USA. All four bridges are girder-type with main girders made of either steel or prestressed concrete. The main girders are simply supported over spans ranging from 26.2 to 43.0 m. The deck is a 200 to 230 mm thickness concrete slab continuous over spans of 2.30 to 3.7 m. Different types and reinforcement ratios of FRP reinforcing bars as well as conventional steel were used as reinforcement for the concrete deck slab. Furthermore, the four bridges are located on different highway categories, which means different traffic volume and environmental conditions (frequency of using de-icing salts). The bridges are well instrumented at critical locations for internal temperature and strain data collection using fibre optic sensors. These gauges are used to monitor the deck behaviour from the time of construction to several years after completion of construction. Three of the bridges were tested for service performance using standard truckloads. The construction procedure, field tests and monitoring results, under real service conditions, showed very competitive performance to concrete bridges reinforced with steel.

INTRODUCTION

Deterioration of concrete structures such as bridges and parking garages due to corrosion of steel reinforcement has limited the service life and increased the maintenance cost of such structures. Concrete bridge decks deteriorate faster than any other bridge component because of direct exposure to environment, de-icing chemicals, and ever-increasing traffic loads. The magnitude of concrete bridge decks cracking and delamination due to corrosion is a major problem when measured in terms of rehabilitation costs and traffic disruption. (1) To overcome the corrosion-related problems, the steel reinforcement should be protected form elements causing corrosion, or be replaced with alternative non-corrosive materials in new structures. One of these alternatives, fiber reinforced polymers (FRP) composite reinforcement, has been used successfully in many industrial applications and more recently has been introduced as concrete reinforcement in bridge decks and other structural elements. The use of non-corrosive FRP composites as reinforcement for concrete bridge decks provides a potential for increased service life, economic, and environmental benefits. (2, 3, 4, 5, 6)

The term FRP describes a group of materials composed of synthetic or organic fibers embedded in a resin matrix (polymer). The most common FRP's targeted to the construction industry are glass FRP (GFRP), carbon FRP (CFRP), and aramid FRP (AFRP). FRP materials in general offer many advantages over the conventional steel, including (1) one quarter to one fifth the density of steel, (2) no corrosion even in harsh chemical environments, (3) neutrality to electrical and magnetic disturbances, and (4) greater tensile strength than steel. (7, 8)

Design guidelines, manuals, and codes for concrete structures reinforced with FRP bars have been recently published. (9, 10, 11, 12) In particular, the new Canadian Highway Bridge Design Code, CHBDC11, includes a new section (Chapter 16) about using FRP composites as reinforcement for concrete bridges. These design guides and codes encouraged and enabled the structures' owners and government agencies such as Ministry of Transportation of Quebec and Vermont Agency of Transportation to implement the FRP composite bar technology in the field.

This paper presents new and innovative field applications of FRP bars as reinforcement for the concrete deck slab of four bridges recently constructed in Canada and USA. The variables in these four bridges, which were constructed in the period from 1997 to 2002, are the type of FRP reinforcing bars (glass or carbon), the reinforcement ratio, the area of the bridge deck reinforced with FRP bars, the location, and category of the bridge (traffic volume and frequency of using de-icing chemicals). Some of these bridges are first of its type to be totally reinforced (top and bottom mats) with Glass FRP bars, and on main highways with heavy traffic. The four bridges are instrumented at critical locations to monitor the behaviour of the bridge from the time the construction starts to several years after the completion of the construction. Three of the four bridges have been tested for service performance using standard truckloads as specified Bridge design codes. (11, 13)
The main objective of these demonstration projects is to implement the technology and design of FRP reinforcing bar and to demonstrate its ability to meet all the requirements for the construction of bridges. Furthermore, it is important to assess the short and long-term performance of FRP reinforcement and to improve/validate the current design guidelines under different service loading and environmental conditions. This paper summarizes the construction details and some results of the field tests and remote monitoring.

STRUCTURAL DETAILS AND DESIGN OF THE BRIDGE DECK SLABS

The tensile properties of the glass and carbon sand-coated ISOROD FRP bars and carbon NEFMAC grids that were used in reinforcing the bridge deck slabs of the four Bridges are listed in Table 1. These FRP bars are manufactured by combining the pultrusion process and an in-line coating process for the outside sand surface. The GFRP bar is made from high-strength E-glass fibers (75% fiber by volume) with a vinyl ester resin, additives, and fillers. The carbon FRP (CFRP) bar (made of 73% carbon fiber by volume, a vinyl ester resin, additives and fillers) has higher tensile strength and stiffness than the GFRP bar (Table 1). The fibers (glass or carbon) give the bar mechanical strength, while the resin matrix (resin, additives, and fillers) provides corrosion resistance in harsh environments.

All four bridges were built with normal-weight concrete. The concrete for the Morristown Bridge had an average 28-day compressive strength of 27 MPa, compared to 47 to 52 MPa for the Joffre, Wotton, and Magog, bridges.

Design forces were determined by a one-way analysis of the slab as well as from the empirical equations. (11, 13) The analysis of the slab was performed assuming a 1.0 m width strip of the transverse slab, continuous over knife-edge supports representing the main girders.

The design of the concrete deck slab for Wotton and Magog Bridges (Quebec, Canada) was originally made with steel bars. Then, the steel reinforcement was replaced by FRP reinforcement according to Clause 16.8.7.11 According to this Clause of the Code, the steel replacement is based on equivalent stiffness for bottom reinforcement layer (ρ_{sB}) and based on equivalent strength for top reinforcement (ρ_{sT}). The FRP reinforcement ratios, ρ_{fB} and ρ_{fT}, were determined using equations 1 and 2.

$$\rho_{fB} = \frac{\phi_s E_s}{\phi_f E_f} \rho_{sB} \qquad \text{for bottom reinforcement} \qquad (1)$$

$$\rho_{fT} = \frac{\phi_s f_y}{\phi_f f_f} \rho_{sT} \qquad \text{for top reinforcement} \qquad (2)$$

Where f_y, E_s, and ϕ_s are the yield strength, the modulus of elasticity, and the resistance factor of steel reinforcement, f_f, E_f, and ϕ_f are the tensile strength, the modulus of elasticity, and the resistance factor of FRP reinforcement. The resistance factor ϕ is taken as 0.75, 0.85, and 0.9 for glass FRP, carbon FRP, and steel bars respectively, Clause 16.5.3 and 8.4.6. (11) The design moments were based on a wheel load of 87.0 kN with the associated load factor of 1.7, Clause 3.5.1(11) a dynamic load allowance of 0.4, Table 3.5.1a (11) and a load combination factor of 0.9, Clause 3.8.4.5.3.(11)

However, for Morristown Bridge (Vermont, USA), crack width, rather than strength and allowable stress limits, was the controlling design factor and determined bar size and spacing for the glass FRP bars in the deck. In this case, the choice of maximum acceptable crack width was 0.5 mm (3, 11). Some geometrical and traffic data are also given in Table 2. The following section presents some construction and reinforcement details of these four bridges.

CONSTRUCTION AND DETAILS OF THE BRIDGES

Installation of FRP bars

The construction crews reacted positively saying that more FRP bars could be handled and placed in less time due to their lightweight. For the bottom reinforcement mat, continuous plastic chairs were placed in the longitudinal direction, spaced at 0.9 m apart, to support the FRP bars and to maintain the required clear concrete cover. While for top reinforcement mat, single chairs spaced at 0.9 m apart in both directions were used. The FRP bars withstood all on-site handling and placement with no problems. It should be noted that there was no need to tie the FRP bars down (no bar floating was noticed during concrete cast). In addition, glass FRP bent bars were successfully installed in the concrete bridge barrier of Wotton Bridge (Type PL-211). More details on the concrete bridge barriers totally reinforced with glass FRP bent bars can be found elsewhere. (14) Furthermore, provisions regarding construction practices (handling, storage, fabrication, and placement of FRP composite materials) are given in ACI 440.1R-03, Chapter 6 (9) and CAN/CSA-S806-02, Section 14 (12).

Joffre Bridge (1997 - Canada)

Joffre Bridge(15), located over St-François River in Downtown Sherbrooke City (Québec), is a girder-type with five spans of continuous steel girders (25.9 m for end-spans and 37.5 m for central spans) totaling 164.4 m and a width of 16.80 m. The 260-mm thick concrete deck slab is continuous over four spans of 3.7 m each and an over hang of 1.0 m. A significant part of concrete deck slab was reinforced with CFRP NEFMAC grids (CFRP bar area 190 mm2; grid 100 mm × 200 mm) in the top mat with a reinforcement ratio of 0.95% and a clear concrete cover of 60 mm. The Joffre bridge was opened to traffic on December 1997 and was tested in 1998 under static and dynamic loads using heavy calibrated trucks.

Wotton Bridge (2001 - Canada)

Wotton Bridge(16) is located in the Municipality of Wotton (on the 6e Rang Ouest, Western Bank, over the Nicolet-Center River in Quebec). The bridge is a girder type with four main girders simply supported over a span of 30.60 m. type with a skew angle of 30 0. The bridge has four main girders simply supported over a single span of 30.60 m. The deck is a 200-mm thickness concrete slab continuous over three spans of 2.65 m each with an overhang of 1.15 m on each side (measured along the skew) as shown in Figure 1a. Standard Type IV AASHTO pre-stressed concrete beams were used as main girders. Curbs, sidewalks, and top layer of the deck slab for half the bridge was reinforced with glass FRP composite bars. Within the same half of the bridge, a 5-m width portion of the bottom layer of the deck slab was reinforced with carbon FRP composite bars. Glass FRP bars were used in all directions (No.16 @ 150 mm, 1.0%, at top main direction and No.16 @ 165 mm, average 0.85%, at top and bottom secondary direction) except in the short direction at the bottom where carbon FRP bars (3 No.10 @ 90 mm - 1.5%) were used as listed in Table 3. The other half of the bridge, including curbs, sidewalks, and top layer of the deck slab as well as the rest of the bottom layer of the deck slab, was reinforced with No. 15M steel bars. The clear concrete cover was 60 and 35 mm at top and bottom, respectively. The construction of the bridge started on July 2001 and it was opened for traffic on October 2001.

Magog Bridge (2002 - Canada)

Magog Bridge(17) is located over Magog River on Highway 55 north (Quebec, Canada) in the vicinity of Sherbrooke City near US/Canadian borders. The total length of the bridge is 83.7 m over three spans. The two end spans are 26.2 m each and the middle one is 31.3 m. The bridge is a girder type with five main steel girders continuously supported over the three spans. The deck is a 220-mm thickness concrete slab continuous over four spans of 2.845 m each with an overhang of 1.352 m on each side.

One full end span (26.2 m), including curbs and sidewalks, were reinforced with FRP bars. The same design criteria as for Wotton Bridge was used. However, the FRP reinforcement ratios were reduced based on the actual required slab thickness and on the test and monitoring results of Wotton Bridge. The same reinforcement amount and configuration as for Wotton Bridge was used but for larger span (2.845 m) and bigger slab thickness (220 mm) as listed in Table 3. The other two spans of the bridge were reinforced with steel bars. The bridge was completed and opened for traffic on October 2002.

Morristown Bridge (2002 - USA)

The Morristown Bridge is located over the Ryder Brook on Route 100 in the town of Morristown (Vermont, USA). The bridge is a girder type, with five main steel girders, integrally cast with the two end abutments over one span of 43.0 m. The deck is a 228.6-mm thickness concrete slab continuous over four spans of 2.364 m each with an overhang of 0.915 m on each side as shown in Figure 1b. This bridge is different than the other three bridges since it was designed based on serviceability criteria (a maximum crack width of 0.5 mm) and the concrete deck slab was totally reinforced with glass FRP bars at top and bottom mats. The design of the deck slab was made according to the AASHTO specifications13 and ACI design9 based on serviceability criteria (a maximum crack width of 0.5 mm). Based on this design approach, the required glass FRP reinforcement bars were: No.19 @ 100 mm and No.16 @ 100 mm in the top and bottom transverse direction, respectively. This difference in top and bottom reinforcement was due to the difference in the concrete cover, 64 mm at top and 38 mm at bottom (Figure 1d). The required glass FRP reinforcement in the longitudinal direction was and No.16 @ 150 mm in top and bottom. As a common practice in bridge engineering and for easy field installation, two identical glass FRP mats, No.19 @ 100 and 150 mm in the transverse and longitudinal directions, respectively, were used at top and bottom (Table 3). There were no splices of the GFRP bars in the transverse direction. However a splice length of 800 mm (about 40 times the bar diameter) in the longitudinal direction was used. It should be noted that there was no pavement for this concrete deck. This is the first bridge deck world wide, of this size and category, which was fully reinforced with glass FRP bars. The total amount of No. 19 (19.1 mm-diameter) glass FRP bars used in the bridge deck slab was 16,775 m. The construction of the Morristown Bridge started on May 2002 and it was opened for traffic on July 2002. Figure 2 shows photos of the four bridges during different stages of construction and testing.

FIELD TESTING

Instrumentation of the Bridges

The bridges are similarly instrumented at critical locations for internal temperature and strain data collection using fibre optic sensors (FOS). Different types of Fabry-Perot and thermocouples FOS were installed on reinforcing bars, embedded in concrete, or glued on the surface of the concrete or steel girders (Fig. 3a). In addition, during testing, deflections of concrete slabs and girders were measured using a system of rulers and theodolites (Fig. 3b). The instrumentation of each bridge was connected to 32-channel FOS data acquisition that is capable of collecting readings from FOS at a rate of up to 10 readings/sec, which is suitable for static testing.

Static and Dynamic Load Test

After the completion of construction, dynamic and static field tests using calibrated heavy trucks were conducted on the bridge in order to evaluate the stress level in FRP reinforcement, concrete deck and steel girders (Fig. 3c). Static and dynamic responses of different components of the bridges are regularly recorded using computer-aided data logging systems since the bridges were opened to traffic. The four Bridges, Joffre, Wotton, Magog, and Morristown Bridges, were tested for service performance.

Similar results were obtained for all bridges in terms of maximum measured strains in FRP and steel reinforcement as well as in concrete. Figure 4 shows comparison between maximum measured strains in reinforcing bars, both FRP and steel, and concrete against truck position along the bridge for Wotton, Magog, and Morristown bridges. In these figures, the zero value on the horizontal axes represent the point at which the midpoint of the second axle (first rear axle) is directly over a given gauge. Maximum strain values do not coincide with the abscissa zero value due to the dual back axle assembly and the influence of the front axle on the strain readings. The strain values depend on the case of loading, namely truck position and path. Therefore, for each graph, the truck path, which gives the maximum strain readings is considered.

In Fig. 4a for Wotton Bridge, it can be seen that a change in strain of only 12 micro-strain was measured in the concrete as the truck moves across the gauge. It is noted here that the concrete gauges were embedded, between two bars, in the deck slab at the same level as top and bottom reinforcement, which is 60 mm and 35 mm, respectively. Using simple bending theory, it can be shown that the tensile strain at the top and bottom surfaces of concrete reached a maximum of 10 and 25 micro-strain, respectively at these gauge locations. These strain values at concrete surfaces of the deck slab are well below the cracking strain of concrete, $\varepsilon cr = 127\ \mu\varepsilon$ (for fc' = 37 MPa and Ec = 29 GPa).

In Figures 4b and 4c, it can be seen that a change in strain of only 4 and 18 micro-strain were measured in the top glass FRP and bottom carbon FRP bars, respectively as the truck moves across the gauge. These strain values were less than 0.16 % of the ultimate strain of the material.

In Figure 4d for Morristown bridge, the single truck recorded the maximum measured strains in bottom and top FRP bars for 31 and 4 micro-strains, respectively. These maximum measured strain values in FRP bars are less than 0.19 % of the material's ultimate strain. The concrete tensile strains can be calculated from the tensile strains measured in the FRP bars. The values of tensile strains at the top and bottom surfaces of concrete slab reached a maximum of 18 and 45 micro-strains, respectively. These values were calculated using the simple bending theory and considering top and bottom concrete covers of 64 and 38 mm, respectively. The strain values at the concrete surface of the deck slab are well below the cracking strain of concrete, εcr = 112 micro-strain (for fc' = 27 MPa and Ec = 24 GPa).

During static tests, deflection of the concrete slabs and steel girders was measured with a theodolite with a system of rulers installed at the mid-span. Figure 4e shows the maximum measured deflection on the steel girders at the bridge mid-span due to truck travel along different paths. Note that truck loading is not evenly distributed on the steel girders. The girder closest to truck travel carries more load than those further away. This was more obvious when the truck traveled over or near the edge girder. The two intermediate girders directly beneath the truck carry 55 to 60% of the total load when one truck is used, while the remaining three girders carry 40 to 45%. These values are 75% to 80% and 20% to 25% when the trucks passed over the edge girder.

REMOTE MONITORING

After the first series of field tests, the DMI-32 FOS data acquisition systems were permanently installed underneath three bridges (Joffre, Wotton, and Magog Bridges). Each data acquisition system was provided with a modem that was hooked up to a phone line to enable data collection from our office at the Université de Sherbrooke. At the office, a computer provided with a modem dials up the phone number and then collect the data and record it automatically using a special software, FISO Technologies Inc. (18) The remote structural health monitoring process will enable us to predict any potential degradation in any particular part of the bridge and provide the adequate maintenance in time. This will decrease the periodical maintenance cost to minimal in terms of rehabilitation needs and traffic interruptions.

For Joffre Bridge, periodical remote monitoring process to collect data from FOS are being carried out since the bridge was opened for traffic on December 5, 1997. The variation of recorded strain with time and temperature through the first five year of the life of the bridge clearly indicates that temperature is the most prominent factor influencing the strain variation in the bridge deck (Fig. 5a). The strains in the FRP bars followed the same pattern as temperature. The minimum temperature and change in strain were -18°C and 520 micro-strain, respectively, which was measured at mid January 2001. While, the maximum temperature and change strain were 33°C and 440 micro-strain, respectively, which was measured at first week of July 2001.

For Wotton Bridge, Figure 5b shows the measured strains in FRP bars and the temperatures of the concrete deck slab a long a duration of one year. Similar results to that of Joffre Bridge were obtained. The minimum temperature and change in strain were -14°C and 420 micro-strain, respectively, which was measured at mid February 2002. While, the maximum temperature and change in strain were 26°C and 360 micro-strain, respectively, which was measured at mid August 2002.

These measured strains due to temperature changed are in the order of 25 to 30 times those measured due to truckloads, however they are still in the range of 3 to 4 % of the ultimate strain of the material. However, these measured strains in Figure 5 are those of the total strains without taking into account the variation in the coefficient of thermal expansion between FOS and FRP bars. The coefficients of thermal expansion for the used FOS and glass FRP bars are the same and approximately equal 4.5 μm/m/0C. While this coefficient equals 0.0 μm/m/0C for carbon FRP bars. (19) Taking this adjustment into account, the maximum measured strains in carbon FRP bars due to temperature changes will be varied by approximately ± 100 micro-strain.

It should be noted that in addition to the remote monitoring a visual inspection process through periodical site visits is being conducted to observe and record the formation or the development of cracks, if any, on the bottom and top surfaces of the deck slab. A similar remote monitoring system is scheduled to be installed for Morristown bridge in Fall 2004.

CONCLUSIONS

Based on the construction details and the results of the field tests, long-term monitoring, and periodical visual inspection, the following conclusions can be drawn:

1. No obstacles to construction were encountered due to the use of the FRP bars in the four concrete bridge deck slabs. The FRP bars withstood normal on-site handling and placement with no problems.
2. The serviceability performance of the concrete deck slab reinforced with FRP bars in terms of strain, cracking, and deflection was very similar to that reinforced with steel bars.
3. During the entire tests, the maximum tensile strain in FRP bars was 16 micro-strain. This value is less than 0.16 % of the ultimate strain of the material.
4. Under service conditions, the measured peak strains in FRP bars were - 520 to +440 micro-strain due to temperature changes of -18 to +33°C through a five-year period of remote monitoring. These values represent around 3 to 4 % of the ultimate strain of the FRP material.
5. The FRP-reinforced bridge decks are very well performing under very harsh environment (de-icing salts, freeze/thaw cycles, elevated temperature, and heavy traffic). No additional or propagation of cracks, if any, were observed under these severe service conditions.
6. These field applications including bridges of different categories and subjected to different environments and service conditions contribute significantly to establish an optimum design for concrete deck slabs reinforced with non-corrosive FRP bars.
7. The construction procedure, field tests and monitoring results, under real service conditions, showed very competitive performance to concrete bridges reinforced with steel.

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Table 1. Properties of FRP reinforcement

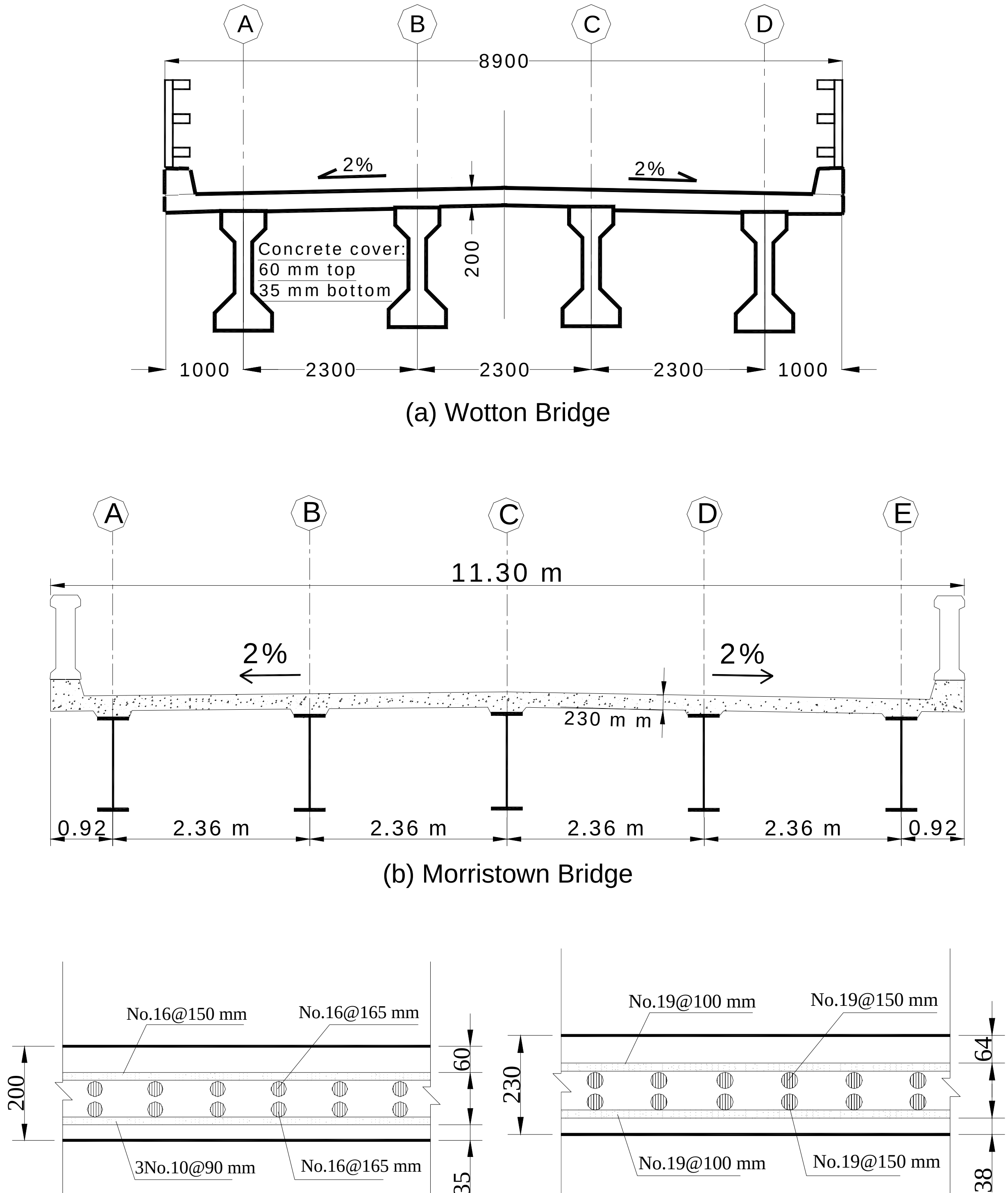
Bar Type	Bar Diameter (mm)	Bar Area (mm^2)	Modulus of Elasticity (GPa)	Tensile Strength (MPa)	Ultimate Strain (%)
CFRP Bars	9.5	71	114 ± 2	1536 ± 31	1.20 ± 0.0
GFRP Bars	15.9	198	40 ± 1	670 ± 26	1.67 ± 0.1
	19.1	285	40 ± 1	597 ± 24	1.49 ± 0.1
NEFMAC CFRP Grids (100×200)	12.7×15.0	190	90± 1	1400 ± 21	1.5 ± 0.1

Table 2. Concrete bridges reinforced with FRP bars

Bridge	Year of construction	Total length × total width (m)	Deck slab			Traffic (Vehicle/day)	Classification
			Thickness (mm)	Span (m)	Reinforcement in the main direction		
Joffre	1997	164.4 × 16.80	260	3.70	Carbon FRP Grids at Top Steel at Bottom	27,000	Urban
Wotton	2001	30.6 × 8.90	200	2.60	Glass FRP bars at Top Carbon FRP bars at Bottom	< 1000	Rural
Magog	2002	83.7 × 14.1	220	2.85	Glass FRP bars at Top Carbon FRP bars at Bottom	35,000	Highway
Morristown	2002	43.0 × 11.30	230	2.36	Glass FRP bars at Top and Bottom	7,000	Urban

Table 3. Reinforcement of the concrete deck slab of Wotton, Magog, and Morristown Bridges

Bridge	Type of bars	Main (transverse) Direction		Secondary (longitudinal) Direction		Overhang (transverse)
		Top	Bottom	Top	Bottom	Top
Wotton	Steel	No.15M@150 mm (1.00 %)	No.15M@150 mm (0.85 %)	No.15M@225 mm (0.67 %)	No.15M@225 mm (0.57 %)	No.15M@ 75 mm (2.00 %)
	FRP	No 16 @ 150 mm (Glass - 1.00 %)	3 No 10 @ 90 mm (Carbon - 1.50 %)	No.16 @ 165 mm (Glass - 0.90 %)	No.16 @ 165 mm (Glass - 0.76 %)	No 16 @ 75 mm (Glass - 2.00 %)
Magog	Steel	No.15M@ 160 mm (0.82 %)	No.15M@ 160 mm (0.70 %)	No.15M@ 240 mm (0.55 %)	No.15M@ 240 mm (0.47 %)	No.15M@ 80 mm (1.64 %)
	FRP	No 16 @ 150 mm (Glass - 0.87 %)	3 No 10 @ 90 mm (Carbon - 1.34 %)	No.16 @ 150 mm (Glass - 0.87 %)	No.16 @ 150 mm (Glass - 0.75 %)	3 No 10 @ 75 mm (Carbon - 1.87 %)
Morristown	FRP	No 19 @ 100 mm (Glass - 1.95 %)	No 19 @ 100 mm (Glass - 1.65 %)	No 19 @ 150 mm (Glass - 1.30 %)	No 19 @ 150 mm (Glass - 1.10 %)	No 19 @ 100 mm (Glass - 1.95 %)



(a) Wotton Bridge

(b) Morristown Bridge

(c) Slab reinforcement of Wotton bridge

(d) Slab reinforcement of Morristown Bridge

Figure 1. Cross-sections of Wotton and Morristown bridges (perpendicular to main girders)

(a) Joffre Bridge (downtown Sherbrooke City)

(b) Carbon FRP grids (Joffre Bridge)

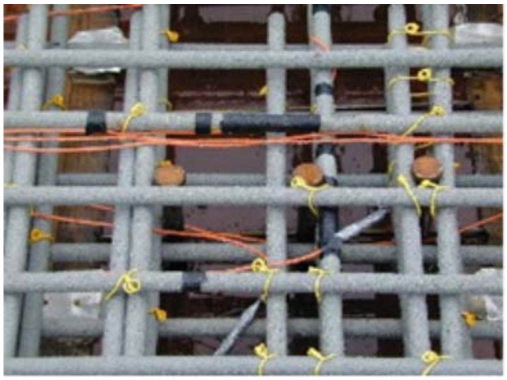
(c) Fiber optic sensors on FRP bars and embedded in concrete (Wotton Bridge)

(d) Close-up for GFRP lap splice (Morristown Bridge)

(e) Casting the bridge deck slab (Morristown Bridge)

(f) FRP reinforcement for the deck slab (Magog Bridge)

Figure 2. Bridges during construction and testing

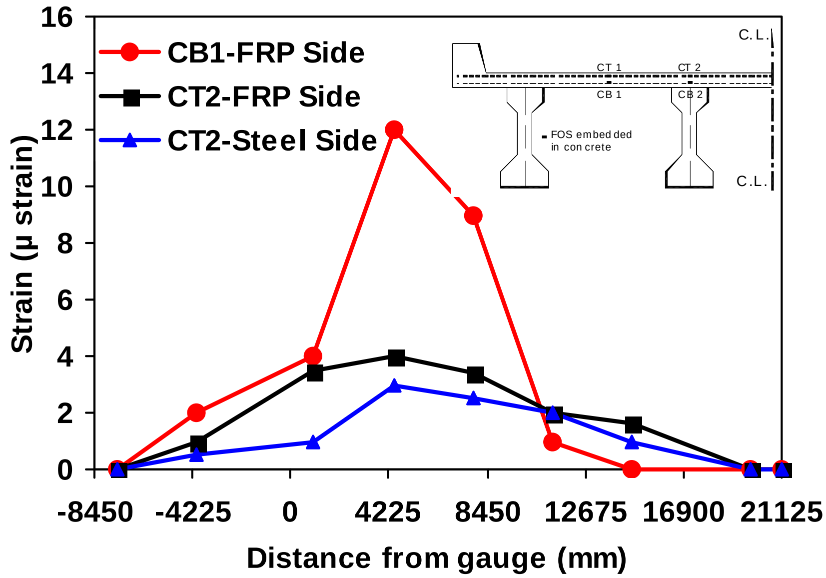


(a)Fiber optic sensors on glass FRP bars

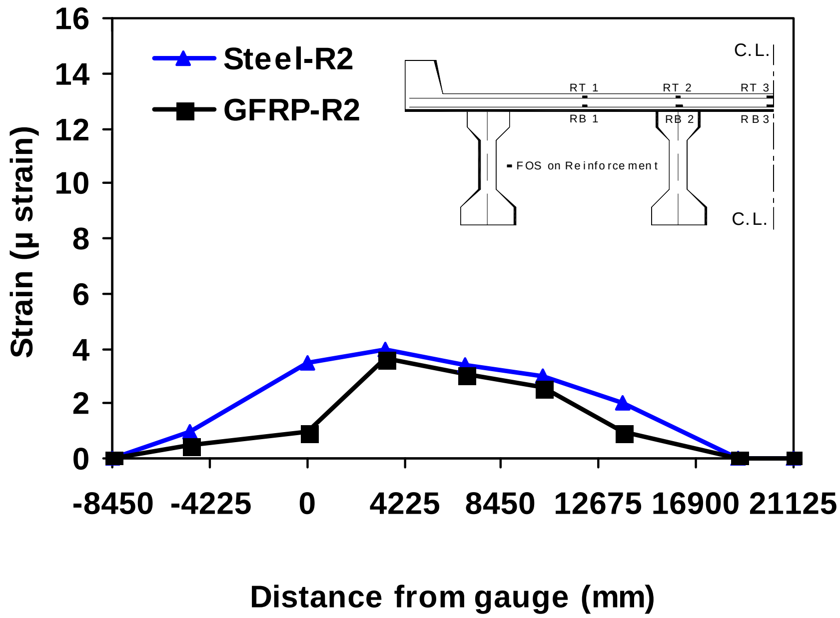
(b)Rulers installed on steel girders

(c) Field testing of Morristown Bridge

Figure 3. Instrumentation and Field testing of the Bridges

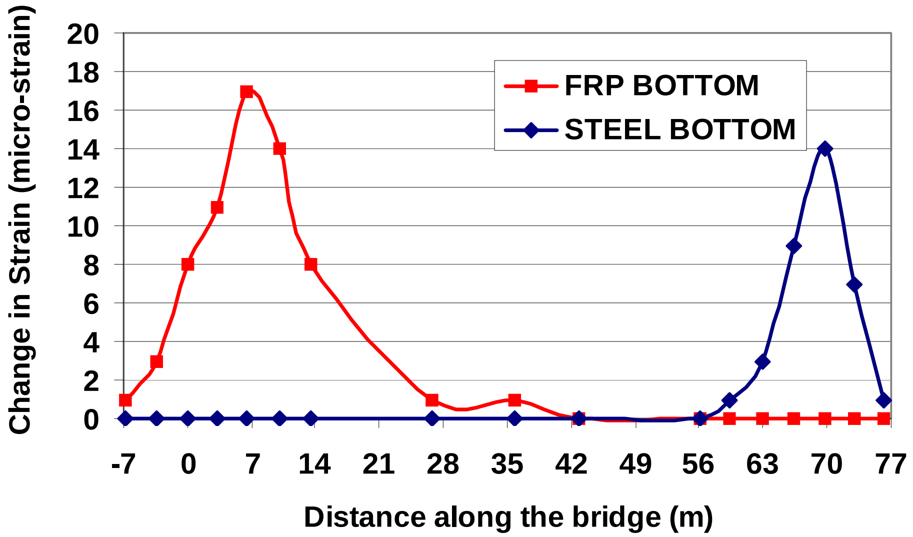


(a) Maximum tensile strains in concrete from embedded FOS (Wotton Bridge)

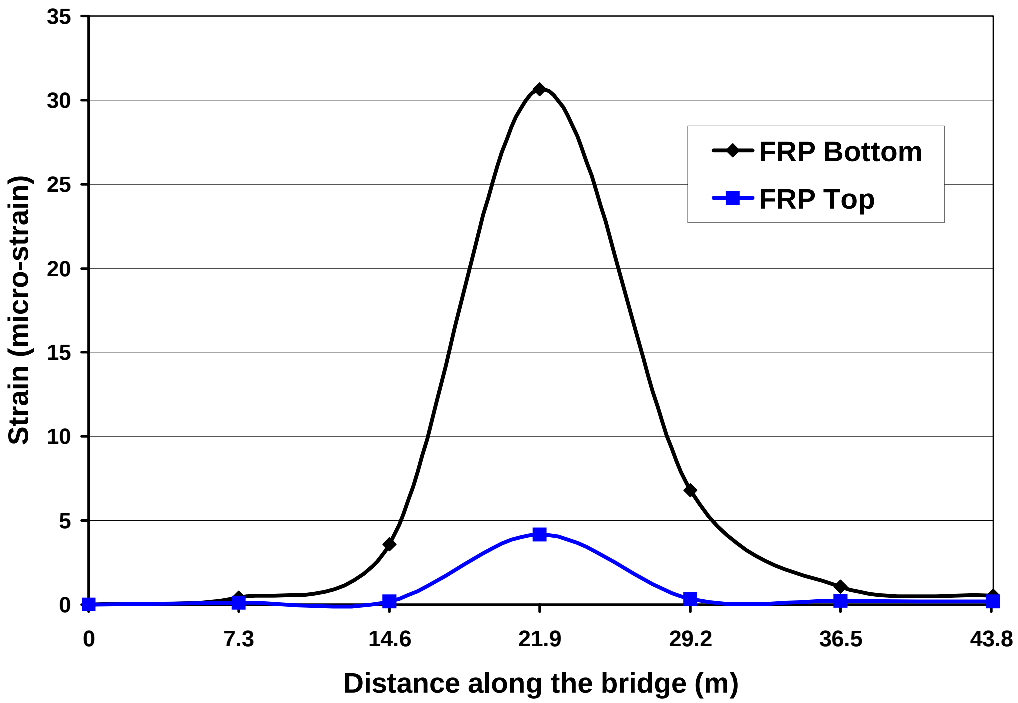


(b) Maximum tensile strains in top reinforcement (Wotton Bridge)

Figure 4. Field test results for Wotton, Magog, and Morristown Bridges

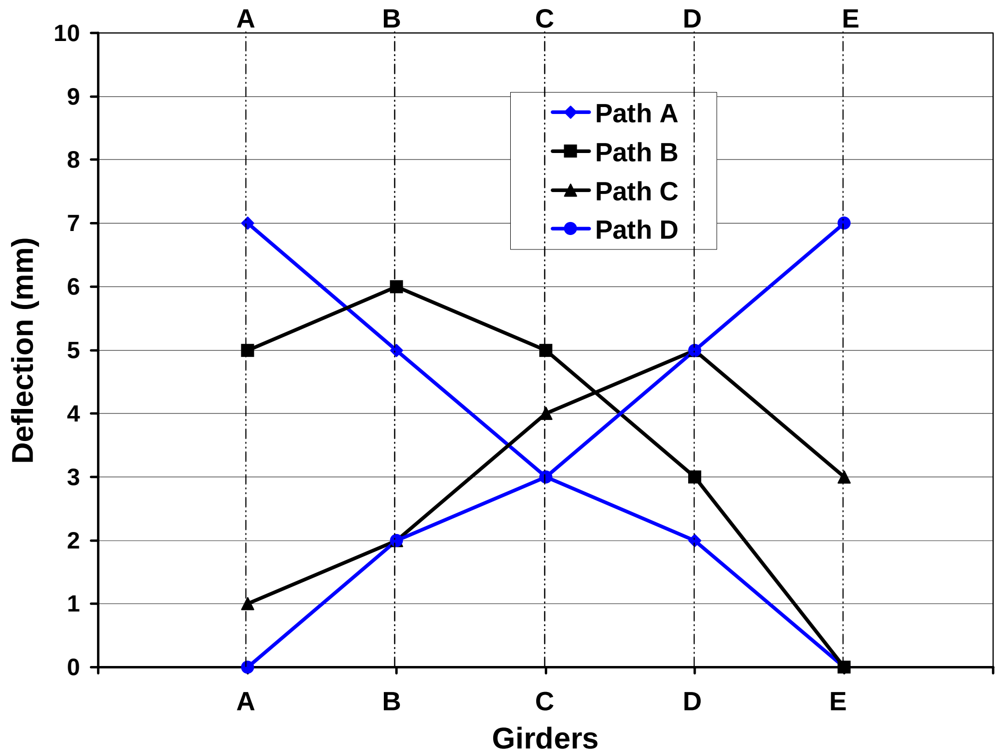


(c) Comparison between strains in FRP and steel bottom reinforcement (Magog Bridge)



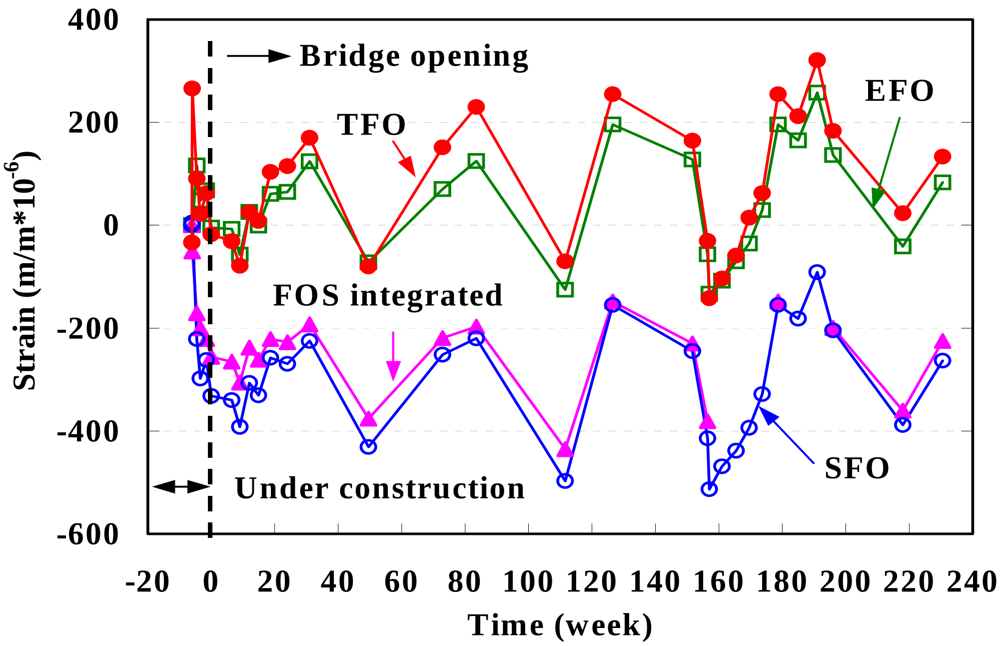
(d) Maximum tensile strains in FRP bars under the wheel loads of one truck (Morristown Bridge)

Figure 4. Field test results for Wotton, Magog, and Morristown Bridges (cont.)

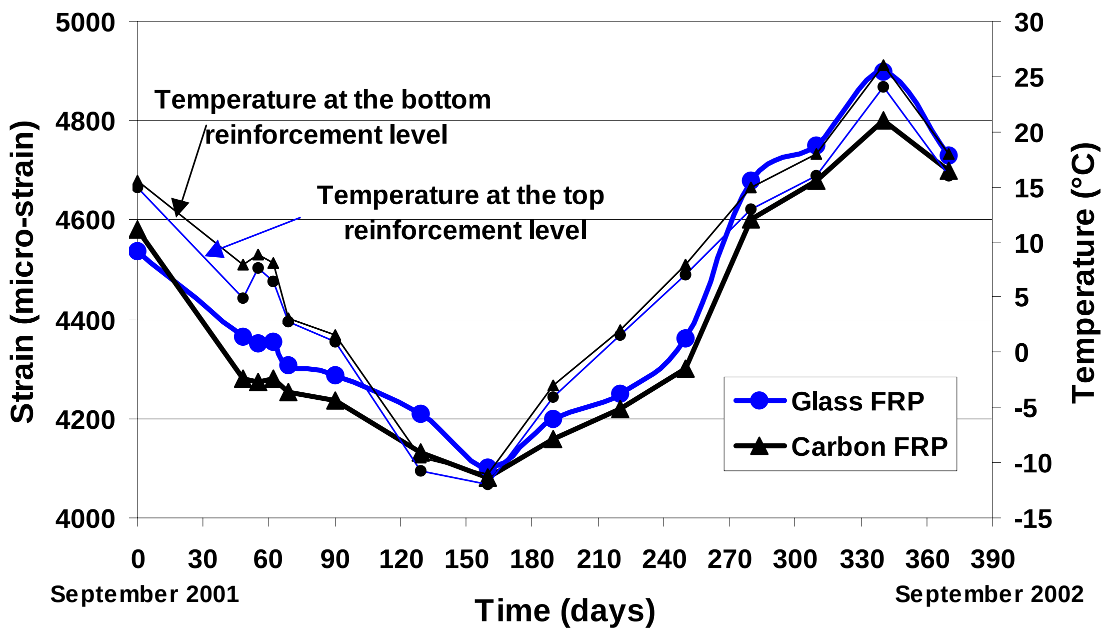


(e) Maximum measured deflection of steel girders (Morristown Bridge)

Figure 4. Field test results for Wotton, Magog, and Morristown Bridges (cont.)



(a) Joffre Bridge



(b) Wotton Bridge

Figure 5. In-service monitoring results