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State-of-the-Art of FRP and SHM Applications in Bridge Structures in Canada

Aftab A. Mufti, University of Manitoba
Kenneth W. Neale, University of Sherbrooke

Abstract

Since 1992 significant efforts have been made in Canada to significantly change the design and construction of bridge structures by developing innovative structures incorporating fiber reinforced polymers (FRP), fiber optic sensors (FOS), and structural health monitoring (SHM). In this paper, some of the innovations that have been implemented through various research and field demonstration bridge projects across Canada are described. This paper also discusses the durability and life cycle costing and engineering (LCC&E) issues that are central to using FRPs and SHM.

Authors

Aftab A. Mufti is currently a Professor of Civil Engineering at the University of Manitoba and President of the ISIS Canada research network. His most recent research is focused on FEM, CAE and CAD/CAM applications in advanced composites, with particular reference to fiber-reinforcement of concrete and wood for bridges. E-mail: muftia@cc.umanitoba.ca.

Kenneth W. Neale is currently Canada Research Chair at the University of Sherbrooke and Vice President of the ISIS Canada research network. His research interests include solid mechanics, materials and structures, and the use of fiber-reinforced polymers (FRP) for civil engineering applications. E-mail: Kenneth.Neale@USherbrooke.ca.

Intelligent Sensing

For many years, engineers have been searching for ways to obtain information on how a structure behaves in-service by incorporating, at the time of construction or subsequently, sensing devices which can provide information about conditions such as strain, temperature, and humidity [11]. The development of such structurally integrated FOSs and intelligent sensing has led to the concept of smart structures.

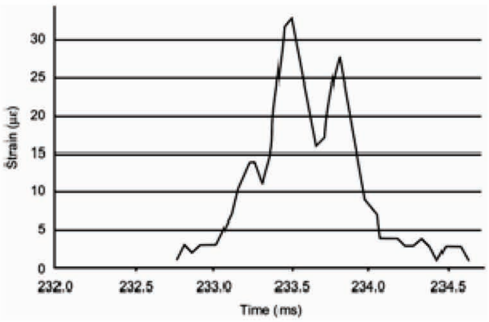
Application of Innovative Technologies in the Field

For infrastructure owners, one of the greatest values of Canadian research lies in its practical applications. Over the past few years, there have been many new opportunities for applying FRP and FOS technologies as is evidenced in the growing number of field demonstration projects underway. Several of the many bridge projects currently being monitored for health in Canada are described in the following sections.

Beddington Trail Bridge, Calgary, Alberta

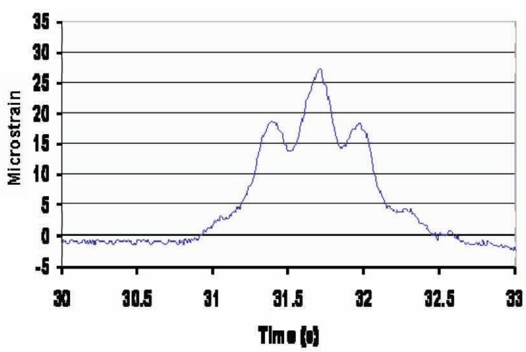
In 1992, the Beddington Trail Bridge in Calgary [9], Alberta, as shown in Figure 1(a), was the first bridge in Canada to be outfitted with FRP tendons and a system of structurally integrated optical sensors for remote monitoring. The bridge opened in 1993. It is significant because, for the group of researchers involved, it confirmed the need for a concerted effort and network that could spearhead transferring this new technology to industry. This led to the creation in 1995 of ISIS Canada, a federally funded Network of Centres of Excellence that encompasses academic and industrial partners across Canada focusing on FRP and SHM R&D and their implementation in practice.

(a)



(b)

(c)



(d)

Figure 1: On-site Monitoring
a) accessing fiber optic junction box – 1999
b) Dynamic FBG Response to three-axle truck load – 1999
c) accessing fiber optic junction box – 2004
d) Dynamic FBG Response to three-axle truck load – 2004

The Beddington Trail Bridge is a 2-span, continuous skew bridge of 22.83 and 19.23-m spans, each consisting of 13 bulb-tee section pre-cast, prestressed concrete girders. Two different types of FRP tendons were used to pretension six precast concrete girders. Carbon fiber composite cables produced by Tokyo Rope of Japan were used to pretension four girders while the other two girders were pretensioned using Leadline rod tendons produced by Mitsubishi Kasei.

Fiber optic Bragg grating (FBG) strain and temperature sensors were used to monitor structural behavior during construction and under serviceability conditions. A 4-channel Bragg grating fiber optic sensor system was used at different locations along the bridge girders that were pretensioned by the carbon FRP. Each fiber optic sensor was attached to the surface of the tendon after pretension to serve as a sensor.

In 1999, the bridge was tested statically and dynamically to assess the durability of the fiber optic sensors. After six years, all FOSs were functioning (Figures 1a and b). This finding validates the view that FOSs are durable and reliable for long-term monitoring.

In November 2004, the bridge was tested again with the same vehicle and weight. Figures 1(c) and (d) indicate that the FBG sensors are durable and are providing accurate results, and that the CFRP is performing as designed in 1993.

Portage Creek Bridge - Strengthening Against Earthquakes & Field Assessment, British Columbia

The Portage Creek Bridge [6] (Figure 2), a relatively high profile bridge that has been classified a Disaster-Route bridge in Victoria, BC, was designed in 1982 by the British Columbia Department of Highways

Figure 2: Portage Creek Bridge

Bridge Engineering Branch. However, it was built prior to current seismic design codes and construction practices and could not resist potential earthquake forces as required by today's standards.

It is a 125 m (410 ft) long 3-span steel structure with a reinforced concrete deck supported on two reinforced concrete piers with abutments on steel H piles. The deck has a roadway width of 16 m (52 ft) with two 1.5 m (5 ft) sidewalks and aluminum railings. The superstructure is supported at the ends and has two intermediate supports along the length of the bridge referred to as Pier No. 1 and Pier No. 2.

The dynamic finite element analysis of the bridge predicts that the two tall columns of Pier No. 1 will form plastic hinges during an earthquake. Once these hinges form, additional shear will be attracted by the short columns of Pier No. 2. Therefore, it was decided that FRP wraps should be used to strengthen the short columns for shear without increasing the moment capacity. The bridge was instrumented with 16 foil gauges, 8 fiber optic sensors and 2 accelerometers (Figure 3) and is being remotely monitored.

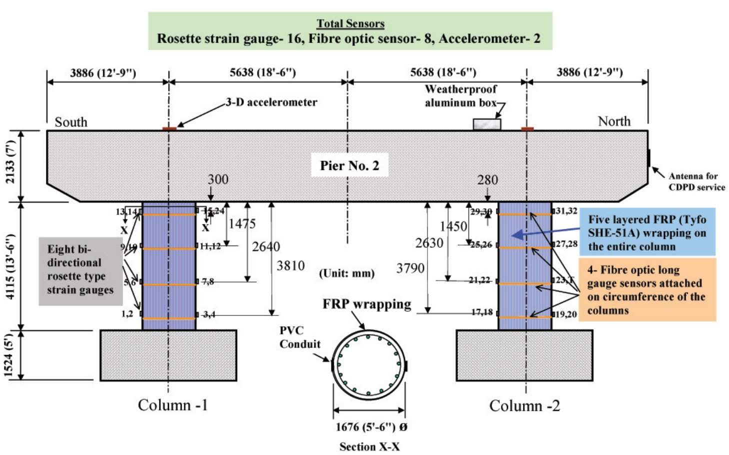


Figure 3: Portage Creek FOS locations

Corrosion-free Bridge Decks

In the design of new highway bridges in Canada, active research is focused on a number of specialty areas, including the replacement of steel reinforcing bars in concrete deck slabs by FRP reinforcement.

The FRPs have perceived disadvantages compared to steel. These are ductility and low thermal compatibility between FRP reinforcements and concrete. The majority of our construction projects in Canada are in non-seismic zones. Ductility is an important characteristic of steel as it allows large deformations and the dissipation of energy. Concrete structures reinforced with FRPs at ultimate loads give large deformations. Therefore, reinforced concrete structures, whether reinforced with steel bars or FRPs, give the same order of deformability. Research is in progress to show that concrete structures with FRPs, if properly designed, can dissipate energy. The design of the proper concrete cover eliminates low thermal compatibility between FRP reinforcement and concrete. It should be noted that glass FRPs have a modulus of elasticity comparable to concrete. This characteristic is believed to be the reason why GFRP reinforced structures perform well in resisting fatigue under dynamic loading.

These concepts have been implemented to develop corrosion-free bridge decks. Several bridge decks have been constructed in Canada and one in Iowa, USA. Some of these bridge decks are described in the following sections.

Salmon River Highway Bridge, Nova Scotia

The first steel-free deck-slab in Canada was cast on the Salmon River Bridge, part of the Trans Canada 104 Highway near Kemptown, Nova Scotia [8]. Construction of the bridge, which consists of two 31-m spans, includes a steel-free deck over one span and a conventional steel-reinforced deck over the other. Internal arching in the slabs helps transfer the loads to the girders.

The steel-free deck contains no rebars. Instead, longitudinal beams or girders support it. The load is transferred from the deck to the supporting girders in the same way that an arch transfers loads to supporting columns. Although steel straps are applied to tie the girders together, because they are not embedded in the concrete they can be easily monitored and inexpensively replaced.

The SHM of the steel-free bridge deck was conducted by installing sensors, as shown in Figure 4. SHM indicates that the load sharing of the Salmon River Highway Bridge is similar to conventional decks, as shown in Figure 5. With no steel inside the concrete (Figure 6), no unnecessary weight is added, meaning thinner deck designs. The steel straps are welded to the top flanges of the girders, thereby resisting any lateral movement. The Salmon River steel-free bridge deck has withstood a number of Canadian winters and

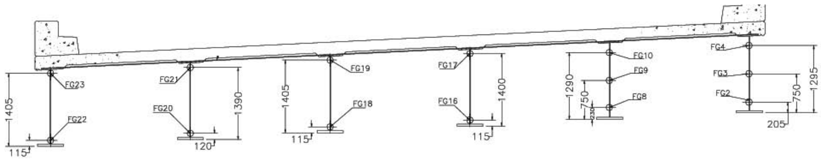


Figure 4: Sensor Locations

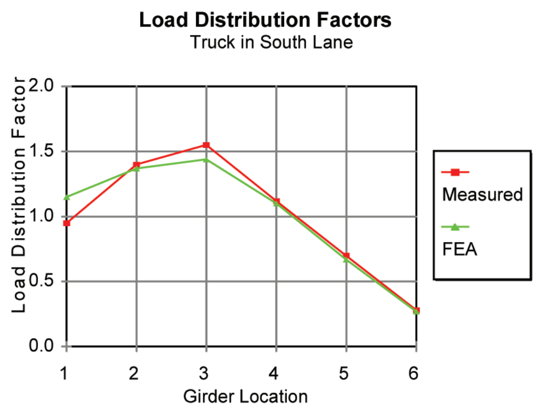


Figure 5: Load sharing of the Salmon River Highway Bridge

Figure 6: Casting of the corrosion-free deck

appears to be defying the conventional approach to building steel-reinforced bridge decks. There are now 10 such corrosion-free bridge decks across Canada.

Second Type of Corrosion-free Bridge Decks

The second type of steel-free deck slab exhibits the same behavior as the first steel-free deck slab, with the exception of the longitudinal crack development at the mid-point between the girders. External steel straps located below the deck provide the structural integrity to the slab. In order to reduce the width of the longitudinal crack that developed on the first type of steel-free decks, researchers at the University of Manitoba [4] concluded that a bottom mat of GFRP reinforcement with a reinforcement ratio of 0.25% was required. In addition, fatigue tests were conducted at the University of Manitoba to replicate actual service life conditions for the deck slab. These tests confirmed that a steel-free deck slab reinforced with a crack-control grid of nominal GFRP reinforcement exhibits a maximum crack width of 0.34 mm, a limit implicitly acceptable by the Canadian Highway Bridge Design Code, CHBDC 2006 [1,4].

North Perimeter Highway Red River Bridge, Winnipeg, Manitoba

This 10-span bridge is 347 meters long and consists of steel plate girders, spaced at 1.8 meters, and a composite, cast-in-place, steel reinforced concrete deck. It is located on the north half of the Perimeter Highway that encircles the City of Winnipeg. Because the Perimeter Highway forms part of the Trans-Canada Highway system, this bridge is subjected to significant daily traffic with approximately 20% being truck traffic.

The one span utilizing the second type of steel-free deck technology was designed and cast using a concrete deck slab thickness of 200 mm. GFRP reinforcement was used for both the top and bottom mats in the internal deck panels. The top and bottom transverse and longitudinal reinforcing were comprised of #3 bars spaced at 200 and 600 mm, respectively (Figure 7). CFRP reinforcement was used as the main reinforcement in negative moment regions for both the vehicular and pedestrian cantilevers. This transverse reinforcing consisted of 2 - #4 bars spaced at 200 mm.

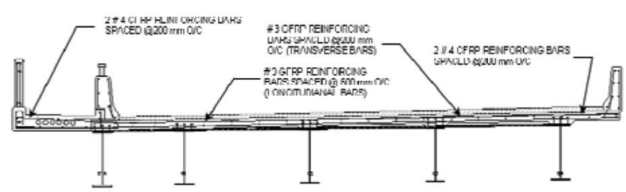
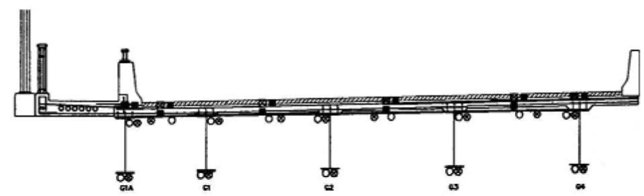


Figure 7: Top and Bottom Transverse and Longitudinal Reinforcement

Transverse confinement of the deck slab was provided by steel straps, measuring 50 mm in width by 30 mm in depth, that have been tack welded to the top flanges of the steel plate girders at a spacing of 1.2 m. To ensure that the steel straps would perform integrally with the deck slab, steel nelson studs were added to the straps in the portion that passed over the girders.

For the Red River Bridge, an integrated SHM system [5] was designed and installed to monitor the components of the steel-free bridge deck slab and to provide data on the stresses in the GFRP reinforcement and transverse steel straps. Stresses in the steel plate girders and the carbon fiber reinforced polymer (CFRP) reinforcement in the negative moment regions for the cantilever sections are also monitored. The system is comprised of a combination of various types of sensors, namely, conventional electric strain gauges, fiber optic Bragg sensors, accelerometers, and thermocouples. A portion of the SHM system for the Red River North Perimeter is shown in Figure 8.



○ Weldable strain gauge on steel strap or girder
⊗ Fiber Optic Bragg Grating on steel strap or girder
☒ Fiber Optic Bragg Grating on FRP reinforcement
■ Electrical strain gauge on FRP reinforcement

Figure 8: SHM System for the Red River Bridge North Perimeter

One major concern of a monitoring system is the enormous quantity of data that is generated, which must be stored in a short period of time. Sensors typically can take up to 100 readings per second, resulting in 8.64 million readings in a single day per sensor. The Red River Bridge contains 64 sensors, which translates into 0.5 billion readings per day. ISIS Canada, in conjunction with IDERS Incorporated, is currently developing an automated system that can be incorporated into the SHM unit. The readings will be scanned for pre-determined strain readings that will initiate a "red flag" notification to the design engineer. The automated system will greatly reduce the time and cost required to review the entire load history of the deck span.

Fatigue Studies of Corrosion-Free Bridge Decks

This section describes the fatigue behavior of a cast-in-place second type of corrosion-free bridge deck. Although cast monolithically, the bridge deck was divided into three segments (A, B and C). Segment A was reinforced according to conventional design with steel reinforcement. Segments B and C were reinforced internally with a CFRP crack control grid and a GFRP crack control grid, respectively, and externally with steel straps. All three segments were designed with an almost equal ultimate capacity so that a direct comparison between the segments under fatigue loading conditions could be made. A performance comparison of all three segments for the first bridge deck under a 60-ton (588 kN) cyclic load is reported in this paper.

Fatigue Testing

Bridge deck details

As stated previously, although cast monolithically, the slab was conceptually divided into three segments: A, B and C (Figure 9).

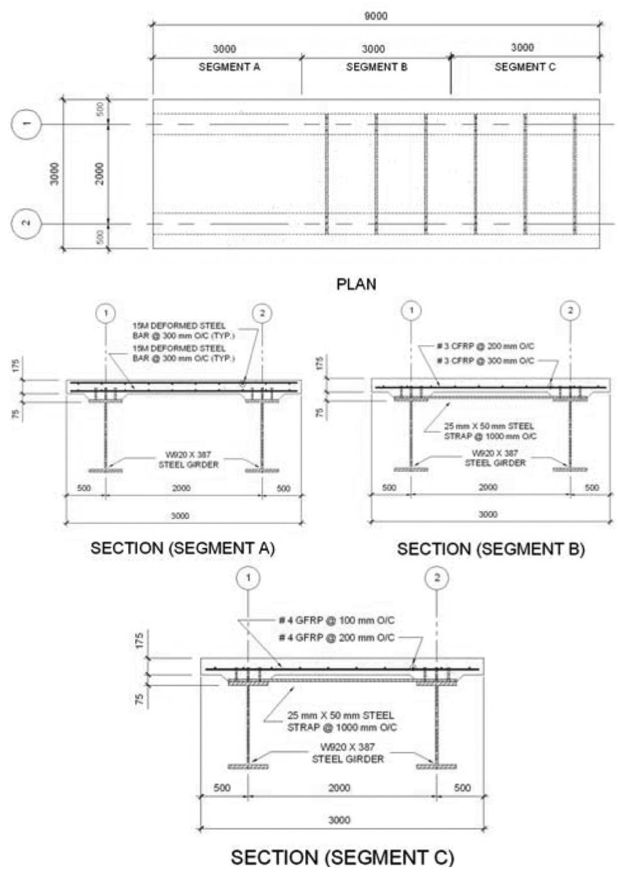


Figure 9: Bridge Deck Reinforcement Details

Figure 10 illustrates the crack width behavior for all three bridge deck segments under the 60-ton or 588 kN load level. The results show that deck Segment A fatigued approximately twenty times as fast as deck Segment C, and deck Segment B fatigued approximately twice as fast as deck Segment C. All three segments failed in fatigue and via punching shear failure.

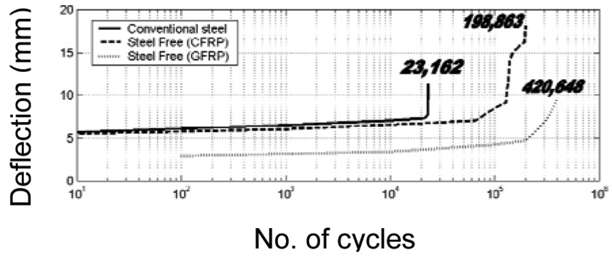


Figure 10: Plot of Deflection versus Number of Cycles at 60 Tons

Cantilever

Although the bridge deck contained an internal panel that was a second type of steel-free bridge deck, only the reinforcement details of the cantilevers are discussed in this section. The bridge deck details are shown in Figure 11.

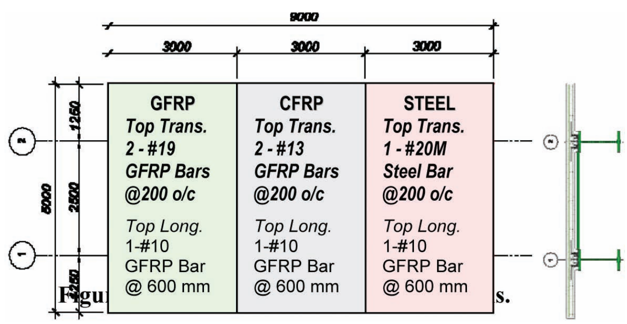


Figure 11: Bridge Deck Cantilever Details

The testing scheme for the cantilevers consisted of six different destructive tests [3]. One of the cantilevers was subjected to three static tests, each of which was conducted on the three different cantilever panels (Figure 12). The static tests were conducted first, in order to determine the ultimate static capacity of each of the cantilevers, before proceeding with the fatigue tests.

GFRP Cantilever Load–Deflection Behavior

The results outlined here deal strictly with the displacement transducer placed at the center of the loading plate (Figure 13). The cantilever failed at an ultimate load of 294 kN and a maximum ultimate deflection of 27.2 mm (Figure 13). The applied load for the fatigue testing was 186 kN or approximately 63% of the ultimate load previously determined by the static test.

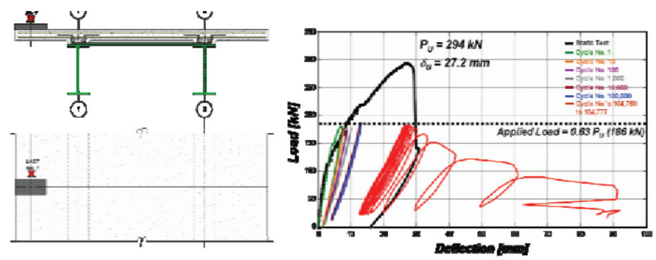


Figure 13: Static & Fatigue Load versus Deflection for GFRP Cantilever

GFRP Cantilever Load–Strain Behavior

A total of 18 electronic strain gauges were installed on the top transverse GFRP bars in order to provide strain data for the top transverse negative moment reinforcement (Figure 14). The maximum strain at the static load of 186 kN occurs over the girder and is 1900 in magnitude, however, at the ultimate of 294 kN the maximum strain has shifted towards the loading plate and is 4282 $\mu\varepsilon$ in magnitude (Figure 14). An important observation is that the strains in the top transverse bar are nearly zero at a distance of 5000 mm from the loaded edge of the cantilever. Looking at the fatigue strain data, it is important to observe that the strain in the top transverse bars remains relatively constant throughout the entire life of the cantilever; however, the strains in the bars are much closer to the applied load, which increases significantly with the increased number of cycles.

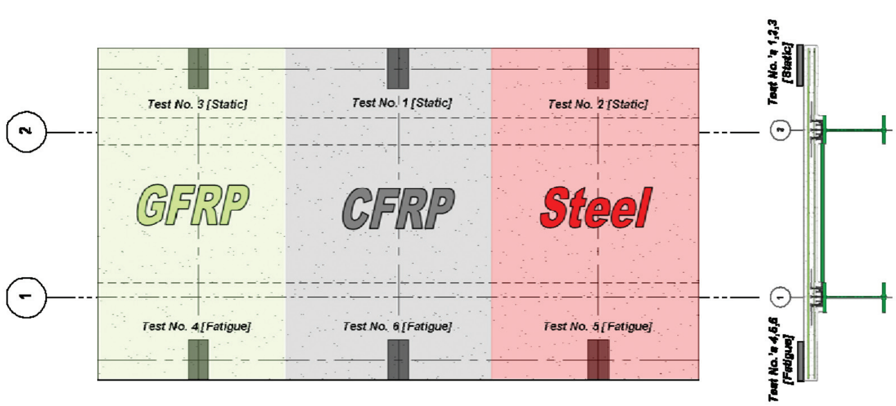


Figure 12: Static & Fatigue Cantilever Testing Scheme

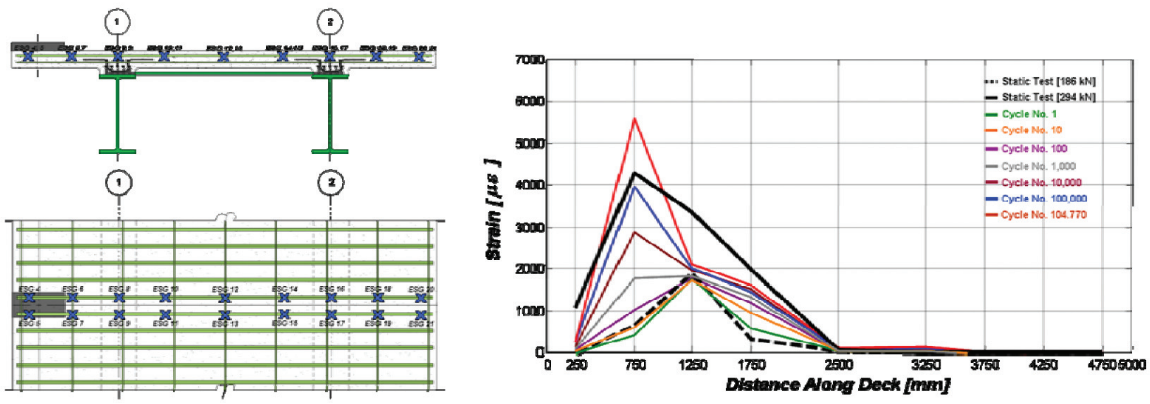


Figure 14: Static & Fatigue Top Transverse GFRP Bar Strain Profile

Studies of Concrete Reinforced with GFRP Specimens from Field Demonstration Projects

The methods used in this study [7] to investigate the degradation of GFRP reinforced concrete are Scanning Electron Microscopy (SEM) and Energy Dispersive X-ray (EDX), Light Microscopy (LM), Differential Scanning Calorimetry (DSC) and Infrared Spectroscopy. To obtain reliable information using such methods, special attention was given to sample preparation. During specimen preparation, the glass fiber can be scratched and microcracks can be induced into the matrix and concrete; the glass fiber can be debonded and the glass and matrix polished surfaces can be contaminated with elements from each other and with elements from the concrete. When such events take place, the interpretation of the test results becomes laborious.

The following is a brief excerpt of the main findings suggested by the results obtained to date from the analysis performed by Mufti et al. [7] at the University of Manitoba on the randomly selected core specimens from the field demonstration projects using the SEM and EDX analyses. Examples of SEM micrographs and EDX spectra on GFRP specimens cored from the Joffre Bridge and Hall's Harbor Wharf as well as from a set of control (unexposed specimens) GFRP specimens showed no deterioration of fibers or resins.

It can be concluded that for the range of conditions of field demonstration projects included in this study, there is no degradation of the GFRP reinforcement. Since the pH of the concrete pore water solution is expected to decrease further with time, it is quite probable that for all practical problems the degradation of GFRP can be considered to be insignificant.

Life Cycle Costing and Engineering

Life cycle performance prediction is the Achilles heel of life cycle costing. This is particularly true when doing life cycle costing for new technologies such as FRPs where empirical observations of the performance through a full life cycle under field conditions do not exist. Such circumstances introduce additional degrees of complexity and uncertainty in predicting life cycle performance. Sparks et al. [10] present a summary of a life cycle costing analysis in which conventional bridge deck rehabilitation techniques are compared with innovative FRP designs within the context of a decision analysis framework. The decision analysis framework provides the capability to explicitly address the complexity and uncertainty inherent in such an analysis. The following section shows a comparison of the LCC&E of the GFRP reinforcement and steel reinforcement.

Nominal data for alternative deck designs are listed in Table 1. The service life of a steel-reinforced deck is set at 50 years. Although uncertainty always surrounds service life estimates, lab and preliminary field results of existing GFRP reinforced concrete decks under similar circumstances suggest GFRP decks will last at least as long as black steel decks [7]. Therefore, a nominal value of 75 years was agreed.

Based on the nominal data, the GFRP deck option proved dominant. The annual worth of life cycle costs (AWLCC) of the steel and GFRP deck options reached about $233,000 and $176,000 CDN, respectively. Thus, the GFRP deck posted life cycle cost savings in the neighborhood of 25%. Yet these results fail to capture the uncertainties surrounding life cycle performance reflected in both service life and repair cycle time estimates. For this reason, sensitivity analysis can provide useful insight regarding the relative influence of cycle time estimates on model results. The sensitivity analysis is described in detail in a paper by Sparks et al. [10].

Three parameters relevant to both deck options were modeled as random variables for the purpose of probabilistic analysis: (i) concrete repair cost, (ii) concrete repair cycle, and (iii) service life. These are summarized in Table 2.

The results of the probabilistic analysis comprise expected value estimates and associated risk profiles. In this case, the expected value of AWLCC under the GFRP and steel reinforced deck design options equal $181,000 and $238,000 CDN dollars, respectively. Moreover, despite the sheer variability in the three selected variables, the dominance of the GFRP option is illustrated by the risk profiles as well. As Figure 15 shows, the GFRP deck design exhibits stochastic dominance over the steel reinforced option.

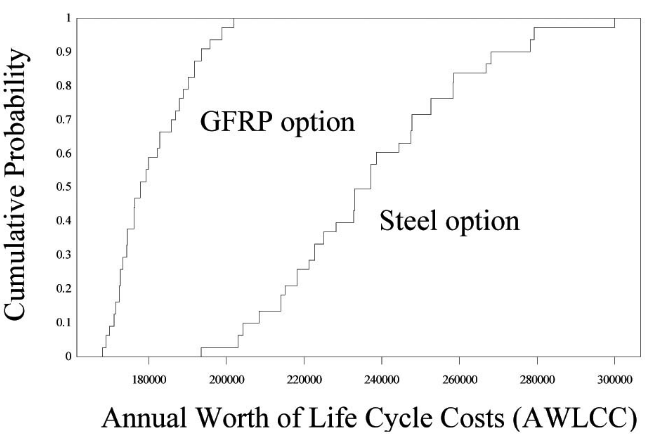


Figure 15: Risk Profiles for Considered Bridge Deck Design Options

Manitoba Floodway Bridges

The research work carried out on the static and fatigue behavior of corrosion-free bridge decks, the monitoring of these decks in the field as well as life cycle costing and engineering led to the decision to build 45,000 square meters of second-generation corrosion-free bridge decks in Winnipeg, Manitoba on the Floodway protecting the city (Figure 16). The details of these bridges are reported by Eden [2].

Figure 16: Manitoba Floodway Bridges

Conclusions and Recommendations

As mentioned earlier, Canadian research and development intends to significantly change the design and construction of civil engineering structures. For changes in design and construction to be accepted, it is necessary that innovative structures be monitored for their health and that codes be established or rewritten to include the use of innovative materials in the construction industry. To assist in achieving this goal, Canada has developed a new approach, which integrates civil engineering and electrophotonics; "Civionics" is the term that has been coined for this. The new philosophy of Civionics must be developed by civil structural engineers and electrophotonics engineers to lend validity and integrity to the process. Civionics will produce engineers with the knowledge to build "smart" structures containing the SHM equipment to provide much needed information related to the health of structures before things go wrong. This approach will, thereby, assist engineers and others to realize the full benefits of monitoring civil engineering structures.

The life cycle costs of any civil structure are a function of its life cycle performance and related treatments (durability of materials, maintenance, repair and rehabilitation). A difficult task at the best of times, reliable life cycle performance forecasts for innovative technologies are particularly hard won. ISIS Canada has initiated research on these difficult but important subjects of LCC&E and durability of materials.

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See Tables next page

Table 1: Nominal data

	Black Steel	GFRP		Black Steel	GFRP
Discount rate:	6.0%	6.0%	Discount rate:	6.0%	6.0%
Service life (years):	50	75	Service life (years):	50	75
Initial Costs			**Maintenance & Repair Costs**		
Design ($):	25,000	35,000	M&R traffic control ($):	75,000	75,000
Traffic control ($):	150,000	150,000	Concrete repair ($):	5,000,000	2,500,000
Deck area (m^2):	6,000	6,000	Concrete cycle (yrs):	25	50
Unit rebar cost ($\$/m^2$):	25	94	Resurface ($):	150,000	150,000
Unit concrete cost ($\$/m^2$):	300	300	Resurface cycle (yrs):	25	25
Install rebar cost ($\$/m^2$):	25	20			
Decommissioning Costs					
	Black Steel	**GFRP**			
Decommissioning ($):	3,000,000	3,000,000			

Table 2: Probabilistic Data for Limited Range of Parameters

		Black Steel			GFRP	
	Low	**Nominal**	**High**	**Low**	**Nominal**	**High**
Service life (years):	40	50	60	50	75	100
Concrete repair ($):	4,000,000	5,000,000	6,000,000	2,000,000	2,500,000	3,000,000
Concrete cycle (yrs):	20	25	30	40	50	60
Corresponding probabilities:	***0.30***	***0.40***	***0.30***	***0.30***	***0.40***	***0.30***