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Finite Element Modeling of Concrete Bridge Deck Slabs Reinforced with FRP Bars

by A. El-Ragaby, E.F. El-Salakawy, and B. Benmokrane

Synopsis: This paper presents the results of a finite element analysis for three different bridges that have been recently constructed and tested in North America. In these bridges, different types of reinforcement (steel and FRP reinforcing bars) were used as reinforcement for the concrete deck slabs. Two bridges, Magog Bridge and Cookshire-Eaton Bridge, are located in Quebec, Canada, while the third one, Morristown Bridge, is located in Vermont, USA. The three bridges are girder-type with main girders made of either steel or prestressed concrete. The main girders were either simply or continuously supported over spans ranging from 26.2 to 43.0 m. The deck was a 200 to 230 mm thickness concrete slab continuous over spans of 2.30 to 2.8 m. Different types, sizes, and reinforcement ratios of glass and carbon FRP reinforcing bars were used. Furthermore, the three bridges are located on different road or highway categories, which mean different traffic volumes and environments. The bridges were tested for service performance using calibrated truckloads. The results of the field load tests were used to verify the finite element model. Comparisons showed that FEM can predict the behavior of such elements. Then, the model was used to investigate the effect of the FRP-reinforcement type and ratio on the service and ultimate behavior of these bridge decks. According to the findings, a proposed reinforcement ratio was recommended and verified using the FEM to meet the strength and serviceability requirements of the design codes.

Keywords: bridges; concrete deck slab; finite element modeling; FRP bars

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INTRODUCTION

In North America, bridges are subjected to harsh environmental conditions such as large fluctuation in temperature, wet-dry and sever freeze-thaw cycles with heavy salt applications. All these factors result in the corrosion of conventional steel reinforcement especially in bridge decks. The expansive corrosion of steel causes cracking and spalling of the concrete decks which limits the service life and increases the maintenance cost of such structures (Yunovich and Thompson 2003).

To overcome the corrosion-related problems, the steel reinforcement should be protected from elements causing corrosion, or be replaced with alternative non-corrosive materials in new structures. One of these alternatives is the fiber reinforced polymers (FRP) composite reinforcement, which recently has been introduced as concrete reinforcement in bridge decks and other structural elements. The corrosion resistance of FRP, in addition to their high-strength and light weight, makes them a promising alternative to traditional steel reinforcement in bridge decks, (Rizkalla et al. 1998; Hassan et al. 1999; Humar and Razaqpur 2000; Yost and Schmeckpeper 2001; Benmokrane and El-Salakawy 2002).

Several codes and design guidelines for concrete structures reinforced with FRP bars have been recently published, (ISIS-M03-01 2001; CAN/CSA-S806-02 2002; ACI 440.1R-03 2003), which encourage the construction industry to use the FRP materials. Furthermore, the first edition of the Canadian Highway Bridge Design Code, CHBDC, (CAN/CSA-S6-00 2000) includes Section 16 on using FRP composite bars as reinforcement for concrete bridges (slabs, girders, and barrier walls). Generally, the design of bridge deck slabs is still based on testing carried out on steel reinforced concrete slabs with some modifications to consider the differences between steel and FRP especially the modulus of elasticity. However, in the last few years, several research was conducted on concrete structures, including deck slabs, reinforced with FRP bars. As a result, Section 16 in the next edition of the CHBDC will be updated taking into account the latest progress made in this field.

Through the NSERC Industrial Research Chair in FRP Reinforcement for Concrete structures, a joint effort with the Vermont Transportation Agency, USA and Ministry of Transportation of Québec (MTQ), Canada, was established to develop and implement FRP reinforcement for concrete bridges. After achieving satisfactory laboratory results on the durability and behavior of FRP reinforcement as reinforcement for concrete elements (Benmokrane et al. 2002; El-Salakawy and Benmokrane 2004), the focus shifted to field applications to push the technology forward. As a result, several bridges, in USA and Canada, were recently constructed with FRP composite bars as reinforcement for the concrete deck slabs. These field applications help to improve the existing design codes and guidelines, establish construction details, and evaluate the performance of FRP reinforcing bars under actual service loading and environmental conditions.

This paper presents the results of a non-linear finite element analysis using the computer program ANACAP–version 2.3 (ANACAP 2004), which was carried out on three bridges, Morristown Bridge (USA), Magog and Cookshire-Eaton Bridges (Canada). These girder bridges (with span-to-depth ratio less than 15) were recently constructed using FRP bars as internal reinforcement for the concrete deck slab. The bridges are different in the span length, thickness of the deck slab, the reinforcement type and ratio, and category of the bridge (traffic volume, frequency of using de-icing chemicals) as shown in Table 1. The finite element model was verified against the field load test results of each bridge thereafter it was used to carry out parametric study to investigate the behavior of bridge decks.

STRUCTURAL DETAILS AND DESIGN OF THE BRIDGE DECK SLABS

Design Criteria

Design forces were determined by a one-way analysis of the deck slab using the flexural design methods (CAN/CSA-S6-00 2000; AASHTO 2000). For the two bridges constructed in Canada, Magog and Cookshire-Eaton Bridges, the design moments were based on a wheel load of 87.5 kN with a dynamic load allowance of 0.4 (Table 3.5.1a) and a load combination factor of 0.9, Clause 3.8.4.5.3, (CAN/CSA-S6-00 2000). For Morristown Bridge, USA, The design moments were based on a wheel load of 80 kN (HS 20-44 standard truck), Clause 3.7.6, using impact factor of 0.3, Clause 3.8.2, (AASHTO 2000).

The design of the concrete deck slab for Magog Bridge (Quebec, Canada) was originally made with steel bars. Then, the steel reinforcement was replaced by FRP reinforcement based on equivalent stiffness for bottom reinforcement layer and based on equivalent strength for top reinforcement, according to CHBDC Clause 16.8.7.11, (CAN/CSA-S6-00 2000).

However, for Cookshire-Eaton and Morristown Bridges, crack width and allowable stress limits were the controlling design parameter and used to determined bar size and spacing for the glass FRP bars in the deck. A maximum allowable crack width of 0.5 mm as well as 15 and 30% of the tensile strength (guaranteed) of the FRP material under sustained and service load levels, respectively, were used.

Properties of Materials

The tensile properties of the steel and FRP bars (carbon and glass) that were used in reinforcing the bridge deck slabs of the three bridges are listed in Table 2. These FRP bars were manufactured by combining the pultrusion process and an in-line coating process for the outside sand surface (Pultrall Inc., Thetford Mines, Québec, Canada). The GFRP bar was made from high-strength E-glass fibers (75% fiber by volume) with a vinyl ester resin, additives, and fillers. The carbon FRP (CFRP) bar was made of 73% carbon fiber by volume, a vinyl ester resin, additives and fillers. All 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 37 and 52 MPa for the Cookshire-Eaton and Magog bridges, respectively. The following section presents some construction and reinforcement details of these three bridges.

Details of the Three Bridges

Magog Bridge (2002 - Canada) -The Magog Bridge on Highway 55 North (Quebec, Canada) spans the Magog River outside of Magog city, which is located close to the US/Canadian border. This girder bridge has a total length of 83.7 m and consists of 5 main steel girders continuously over three spans. The two end spans are 26.20 m; the middle one at 31.30 m. The deck consists of a 220-mm thick concrete slab continuous over five girders spaced at 2.85 m with an overhang of 1.35 m on each side. The clear concrete cover is 35 and 60 mm at bottom and top, respectively (Fig. 1). One full end span (26.20 m), including curbs and sidewalks, was reinforced with FRP bars. Carbon FRP bars (3 No.10 @ 90 mm - 1.5%) were used as bottom transverse reinforcement while Glass FRP bars were used in all the remaining directions (No.16 @ 150 mm, 1.0%, in top transverse direction and No.16 @ 165 mm, average 0.85%, in top and bottom longitudinal direction). The other two spans of bridge were reinforced with two identical top and bottom mats of galvanized steel bars (No.15M @ 160 mm and No.15M @ 240 mm in the transverse and longitudinal directions, respectively). The bridge was completed and opened to traffic on October 2002. More details on the construction and testing of this bridge can be found elsewhere (El-Salakawy and Benmokrane 2003).

Morristown Bridge (2003 – 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 43.00 m. The deck is a 228 mm thickness concrete slab continuous over four spans of 2.36 m each with an overhang of 0.90 m on each side as shown in Fig. 2a. The concrete deck slab was totally reinforced with glass FRP bars at top and bottom mats. Two identical glass FRP mats, No.19 @ 100 and 150 mm in the transverse and longitudinal directions, respectively, were used at top and bottom as shown in Fig. 2b. This is the first bridge deck world wide, of this size and category, which was fully reinforced with glass FRP bars. The construction of the Morristown Bridge started on May 2002 and it was opened to traffic on July 2002. More details on the construction and testing of this bridge can be found elsewhere (Benmokrane et al. 2005).

Cookshire-Eaton Bridge (2004 – Canada) -The Cookshire-Eaton Bridge is a girder type bridge that crosses the Eaton River at the centre of Cookshire town. The bridge has five main girders (standard AASHTO Type IV prestressed concrete beams) continuously over two spans of 26.04 m. The deck is a 200 mm thick concrete slab continuous over five girders spaced at 2.70 m with an overhang of 1.40 m on each side, (Fig. 3b). One full span including the deck slab, curbs and sidewalks, was totally reinforced with GFRP bars. No.19 GFRP bars at 75 mm and 100 mm in the top and bottom transverse directions, respectively, were used. No.16 GFRP bars at 150 mm were used in the longitudinal direction at the top and bottom. The other bridge span, including the deck slab, curbs, and sidewalks, was reinforced with two identical mats of No. 15M steel bars spaced at 150 mm and 225 mm in the transverse and longitudinal directions, respectively, as shown in Fig. 3c. The clear concrete cover was 60 and 38 mm at the top and bottom, respectively. The construction of the Cookshire-Eaton Bridge started on September 2003 and it was opened to traffic on February 2004. More details on the construction and testing of this bridge can be found elsewhere (El Ragaby et al. 2004).

Field Load Testing of the Bridges

Instrumentation of the bridges - During construction, the three bridges were well instrumented at critical locations for internal strain data collection using Fabry-Perot fiber optic sensors (FOS), as shown in Figure 4. The bridges were similarly instrumented at one or two, locations along the bridge (at one-quarter and/or midway of each span). FOS were installed on reinforcing bars, embedded in concrete, or glued on the surface of the concrete or steel girders to monitor strains of both concrete and reinforcing bars (steel and FRP). The instrumentation of each bridge was connected to 32-channel FOS data acquisition system that is capable of collecting readings from FOS at a rate of 10 and 100 readings/sec, which is suitable for static and dynamic testing, respectively.

Static and dynamic load test - After the completion of construction, dynamic and static field tests using calibrated heavy trucks were conducted on the bridges to evaluate the stress level in FRP reinforcement, concrete deck and girders. Different paths and truck loads that expected to produce maximum stress were used. Also different combinations of single or double trucks were employed, as shown in Fig. 5. During static tests, deflection of the concrete slabs and steel girders was measured using a theodolite and a system of rulers installed at the mid-span of each bridge.

FINITE ELEMENT MODELING OF THE BRIDGE

The analytical model used in this investigation was developed using the non-linear finite element analysis program ANACAP–version 2.3 (ANACAP 2004). The concrete material model is based on a smeared cracking methodology where cracks are assumed to form in three different directions perpendicular to the principal tensile strain directions in which the cracking criterion is exceeded. The model also accounts for the concrete non-linear properties such as residual tension stiffness, shear retention, reduction in shear stiffness due to cracking. The compressive stress-strain curve is followed up to the ultimate strength and into the strain softening regime through modified Drucker-Prager yield criteria (combined Mohr-Coulomb and Von Mises criterion). The reinforcement is modeled as individual sub-elements within the concrete elements. Reinforcing bar sub-elements stiffness is superimposed on the concrete element stiffness in which the rebar resides.

The validity of the analytical model to investigate the behavior of bridge decks reinforced with FRP bars was verified against the results of an experimental research program that is being conducted by the authors at the Université de Sherbrooke. Comparisons showed that the FEM can be used to understand and predict the behavior of such element (El Ragaby et al. 2005).

Finite Element Modeling for the Full-Scale Bridge

For Each bridge, only one full span was modeled. The deck slab was divided into three equivalent segments in the longitudinal direction. The middle segment was divided into a fine mesh with small size of elements in both longitudinal and transverse directions to be able to apply any combinations of loads at any position within this segment with an acceptable accuracy. In the other two segments of each span, away from loading zone, a gross mesh with larger element size was used to reduce the time required to run the analysis. The thickness of the slab was divided into three elements with unequal thicknesses. The supporting steel or concrete girders and the corresponding intermediate cross-girders were modeled with their actual dimensions, location, and shape as shown in Fig. 6. Iso-parametric brick elements with 20-Nodes each were used to model the deck slab and girders. Reinforcement was modeled as individual sub-elements; rebar elements, embedded in concrete with their actual material properties, spacing, and size.

The truck axle (2 wheel-loads) was modeled as specified by the CHBDC with a wheel footprint of 250 x 600 mm and transverse spacing between wheels of 1800 mm (CAN/CSA-S6-00 2000). The load was applied as a uniformly distributed pressure over the loaded area through an incremental process. The finite element mesh was dimensioned to be able to locate the different truck paths in identical manner to the field testing. For each path, only one truck stop that gives the maximum strain readings, (exactly at the instrumented section) was considered.

COMPARISONS BETWEEN MEASURED AND FEM PREDICTIONS

Figure 7 shows the maximum measured strain values, from FOS, during the entire field load tests on the three bridges compared to those predicted by the FEM. It can be seen that the maximum measured tensile strains in the bottom transverse GFRP bars for Cookshire-Eaton and Morristown Bridges were about 30 micro-strain while for the bottom transverse CFRP bars for Magog Bridge was about 20 micro-strain. Using the FEM, these values were 26, 28, and 22 micro-strain, respectively. Also the maximum measured tensile strains in the top transverse bars for the three bridges were 16, 8, and 9 micro-strain while the FEM predicted ones were 17, 7.5, and 12 micro-strain, respectively. This indicates that the FEM can be used to predict the behavior of bridge deck slabs reinforced with different types of FRP bars with an acceptable degree of accuracy (approximately 87% to 94%).

Since the maximum wheel load during field testing was less than the specified wheel load at service load level (110.25 kN, Clauses 3.5 and 3.8 of CHBDC), the FEM was used to check the strains in the FRP bars at that load level. As shown in Figure 7, the expected tensile strain under service load level in the bottom transverse GFRP bars of Cookshire-Eaton and Morristown Bridges is about 75 micro-strain and for the bottom transverse CFRP bars of Magog Bridge is about 58 micro-strain. These strain values are less than 0.5% of the ultimate strain of the used CFRP and GFRP bars.

To investigate the ultimate load capacity of the bridges using the FEM, the load was applied at middle of the exterior span for both the steel and FRP reinforced spans for Magog and Cookshire-Eaton Bridges. Figure 8 shows the relationship between the applied load and the maximum deflection at mid-span (center of loading). It can be seen that both the steel and FRP reinforced spans has similar ultimate capacity of 925, and 890 kN for Magog and Cookshire-Eaton Bridges, respectively. This is approximately 4 times the ultimate design load (208.25 kN), with a maximum expected deflection of about 10.0 mm. Also at service load level, the maximum deflection at mid-span between girders is 1.0 mm, which is less than the allowable deflection by the CHBDC (CAN/CSA-S6-2000).

EFFECT OF FRP REINFORCEMENT RATIO

Based on the results of the previous analysis for the three bridges, the large safety margins compared to code requirements at serviceability limit states can be noted. The analysis carried out in this research and by other researchers, (Ospina et al. 2003), indicates that the behavior of the FRP reinforced deck slab is controlled primarily by punching shear rather than flexure. Therefore, the top reinforcement ratio has a minor effect on the punching shear capacity of continuous bridge deck slabs. Glass FRP bars with reinforcement ratio of 1.2% in the bottom transverse reinforcement and 0.6% in the remaining three directions were previously recommended by the authors and other researchers (Hassan et al. 2000; El-Salakawy and Benmokrane 2003). It is interesting to use the FEM to evaluate the serviceability and ultimate performance of the previous three bridge deck slabs using this reduced reinforcement ratio for the top and bottom GFRP reinforcing bars.

Figure 9 (a) shows the load-strain relationship for the deck slab of Magog Bridge utilizing three different reinforcement types and ratios. The proposed reinforcement ratio of GFRP bars consists of No.19 GFRP at 135 mm in bottom transverse direction and No.16 GFRP bars at 185 mm in the remaining directions. There was insignificant effect on the ultimate capacity of the deck slab and both reinforcement strain, and deck slab deflection at service load limit. At failure, the strain in the proposed GFRP reinforcement ratio is about 200% higher than that of actual CFRP reinforcement ratio but still less than 60 % of the ultimate strain of the used GFRP bars.

It was found also that reducing the top and bottom transverse reinforcement ratio by 37%, for Cookshire-Eaton Bridge, has insignificant effect on the ultimate strain, deflection, and reinforcement strain values at service load level as shown in Fig. 9 (b). The predicted punching shear carrying capacity is 0.6 and 4.6% less than that of the bridge deck slab reinforced with the actual GFRP (1.80 %) and steel (0.85 %) reinforcement, respectively. At failure, the strain in the proposed GFRP reinforcement ratio is about 24% higher than that in the actual GFRP reinforcement ratio (1.8%). However, this strain value is still less than 3% of the ultimate strain of the used GFRP bars.

Figure 10 (a) shows comparisons of cracking patterns for the actual deck design (with both steel and CFRP) for Magog Bridge, and the proposed GFRP reinforcement ratio at service load level. It can be noticed that more and closer cracks exist due to the difference in modulus of elasticity and bar spacing of both the original CFRP and the proposed GFRP reinforcement. In case of Cookshire-Eaton Bridge, it was noticed also that using the proposed GFRP reduced reinforcement ratio results in similar crack pattern as the actual GFRP reinforcement ratio as shown in Fig. 10 (b). Both FRP reinforcement ratios resulted in more cracks compared to steel reinforcement. This was expected due to the significant difference in modulus of elasticity between steel and FRP reinforcement. This means that the FEM can predict the cracking pattern and initiation for both FRP and steel reinforced deck slabs with good accuracy.

CONCLUSIONS

Three girder type bridges (Magog, Cookshire-Eaton, and Morristown Bridges) have been recently constructed and tested in Canada and USA. The concrete deck slabs of those three bridges were reinforced with glass and carbon FRP bars. A non-linear finite element analysis software (ANACAP 2004) was used to analyze the behavior of the three bridges under service and ultimate loading conditions. Based on the results of the field tests and the FEM (concrete bridge deck slabs supported on girders with span-to-depth ratio less than 15), the following conclusions can be drawn:

1. The FEM is capable of predicting the behavior, the carrying capacity, and the mode of failure of bridge deck slabs reinforced with FRP or steel reinforcement.

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. The proposed GFRP reinforcement ratios (1.2% in the bottom transverse reinforcement and 0.6% in the remaining three directions) are adequate for satisfying the serviceability and the strength criteria required by the Canadian Highway Bridges Design Code. These proposed reinforcement ratios are in good agreement with the ratios recommended by Section 16 of the next edition of the CHBDC.

4. The maximum tensile stresses in the bottom transverse GFRP bars at service load levels are less than 5.0 MPa, which is less than 1.0% of the ultimate tensile strength of the GFRP bars. Therefore, the phenomenon of creep-rupture of GFRP bars is not a concern.

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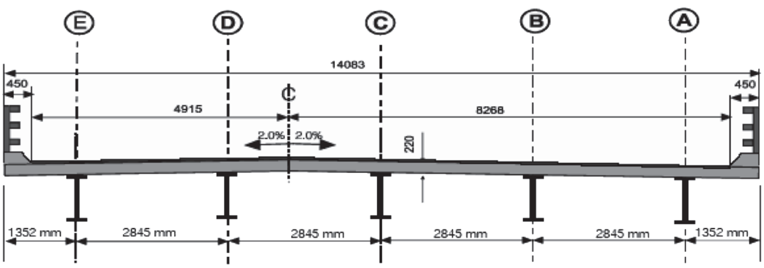
Table 1 – Concrete Bridges Reinforced with FRP

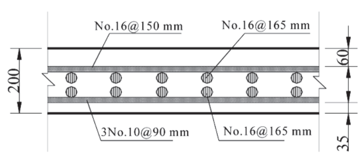
Bridge	Total length × total width (m)	Deck slab			Traffic (Vehicle/day)	Class
		Thickness (mm)	Span (m)	Reinforcement in the main direction		
Magog	83.7× 14.1	220	2.85	Glass FRP bars at Top Carbon FRP bars at Bottom	35,000	Highway
Morristown	43.0×11.30	230	2.36	Glass FRP bars at Top and Bottom	7,000	Urban
Cookshire-Eaton	52.85x13.6	200	2.70	Glass FRP bars at Top and Bottom	7,000	Urban

Table 2 - Properties of GFRP reinforcement

Bar Type	Bar Diameter (mm)	Bar Area (mm2)	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	44 ± 1	755 ± 26	1.67 ± 0.1
	19.1	285	42 ± 1	612 ± 24	1.59 ± 0.1
Steel	15.9	200	200	$f_y = 480$	$\varepsilon_y = 0.2$



(a) Cross section



(b) FRP reinforcement

Figure 1 - Dimensions and Reinforcement of Magog Bridge.

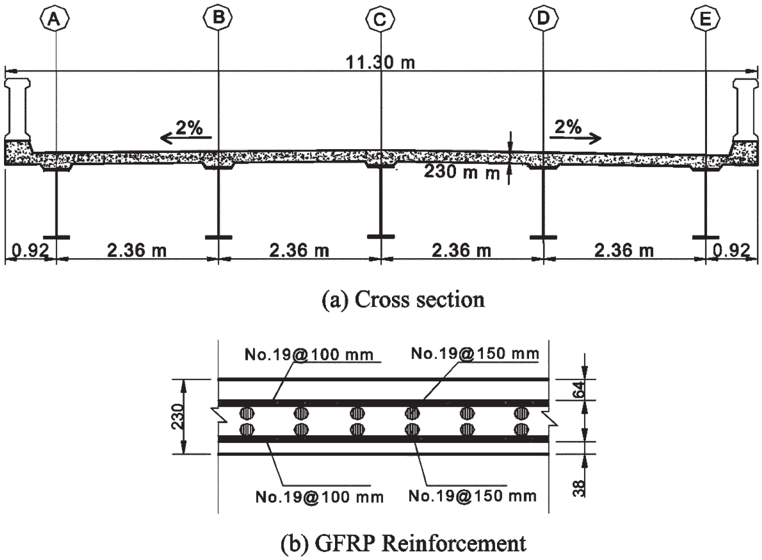


Figure 2 - Dimensions and Reinforcement of Morristown Bridge.

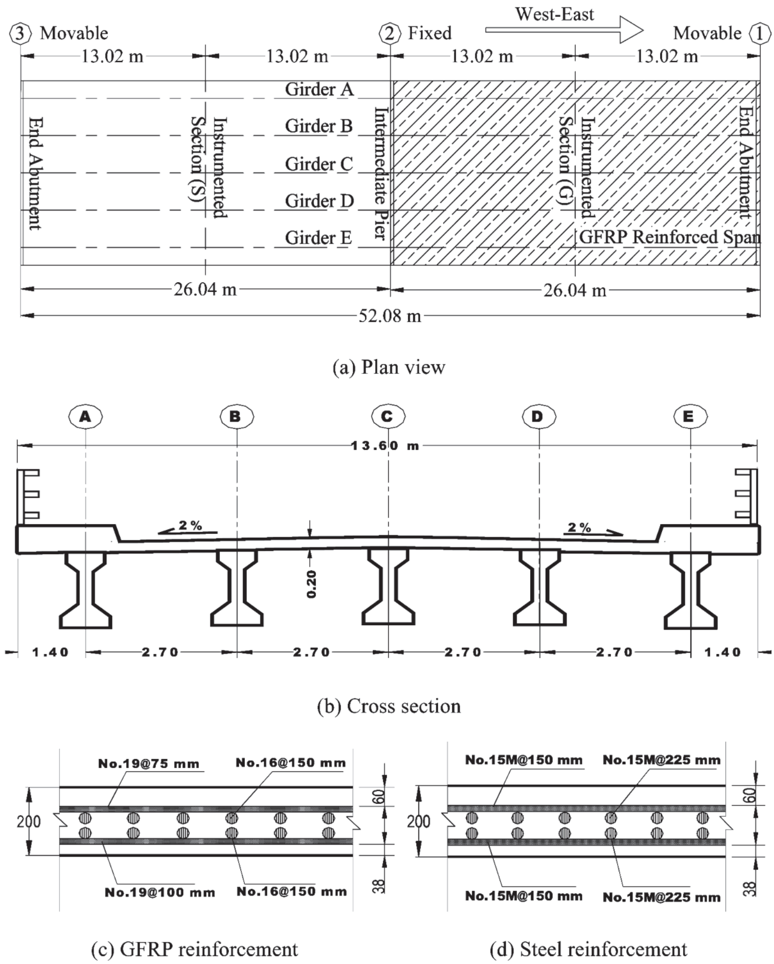
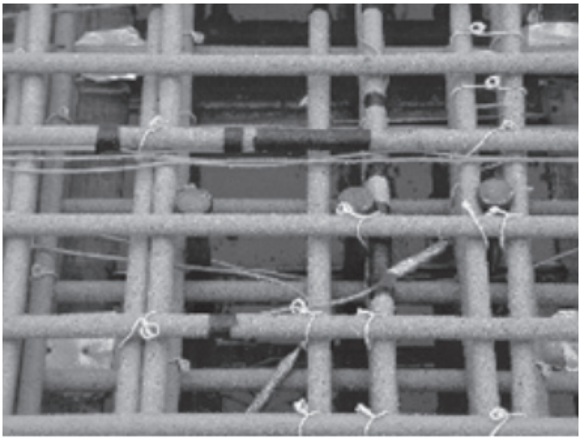
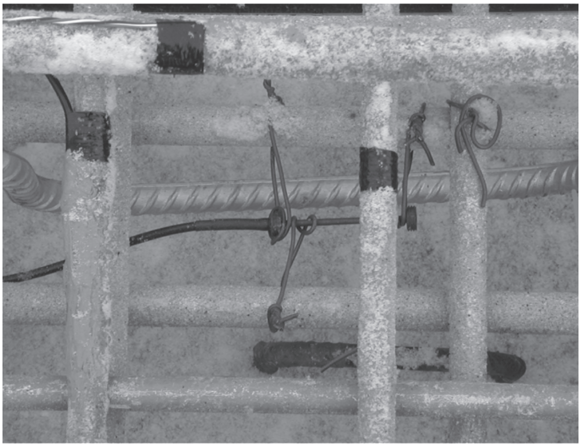


Figure 3 - Dimensions and Reinforcement of Cookshire-Eaton Bridge.

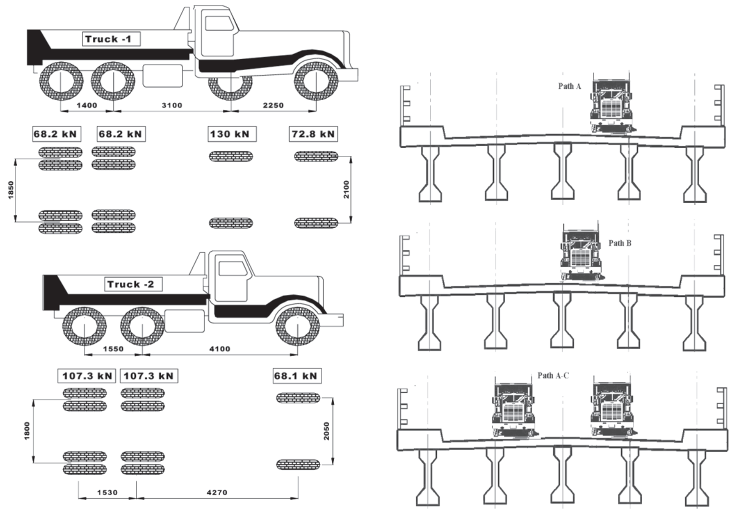


(a) FOS glued on FRP reinforcement



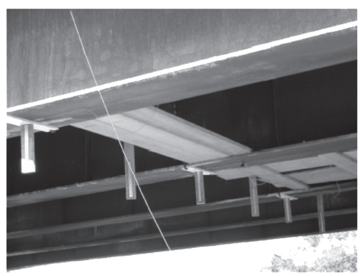
(b) FOS embedded on concrete

Figure 4 - Instrumentations for the bridges.



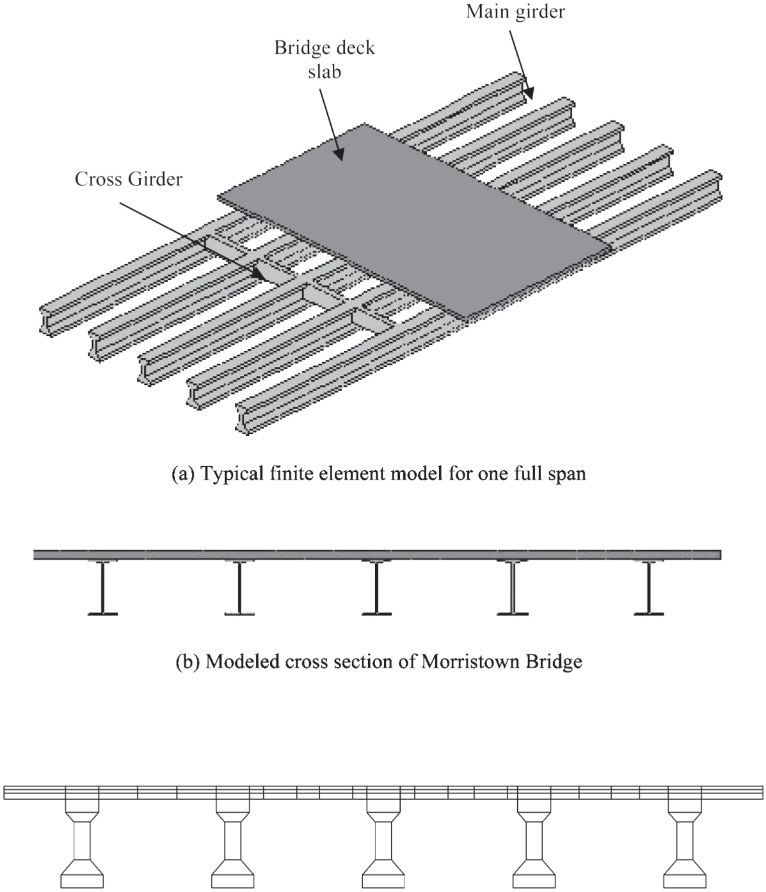
(a) Wheel loads and truck paths for Cookshire-Eaton Bridge

(b) Magog Bridge during field test



(c) Rulers for deflection measurements

Figure 5 - Field load test of the bridges.



(a) Typical finite element model for one full span

(b) Modeled cross section of Morristown Bridge

(c) Finite element mesh of the deck slab and main girders (Cookshire-Eaton Bridge)

Figure 6 - Finite element modeling of the bridges.

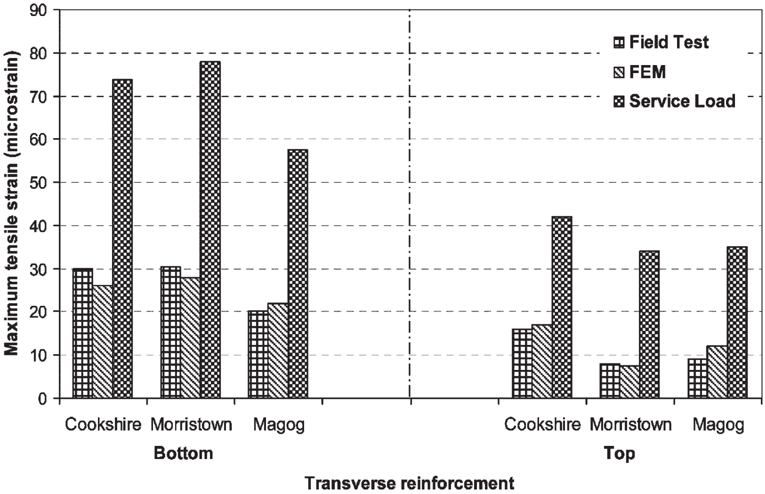
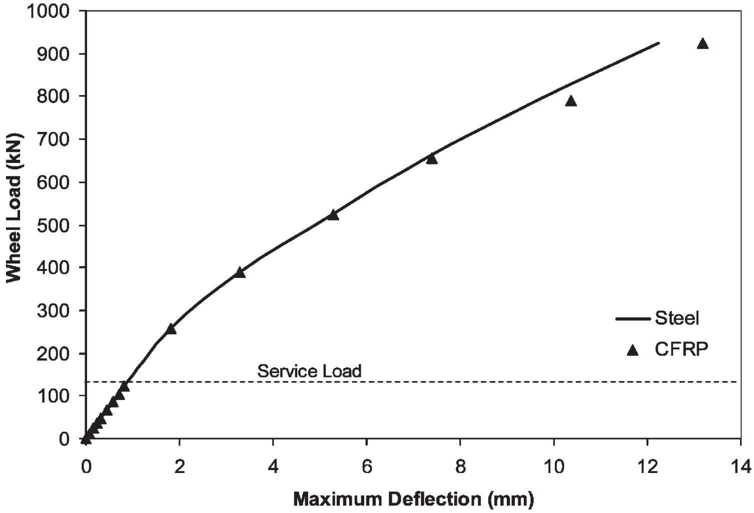
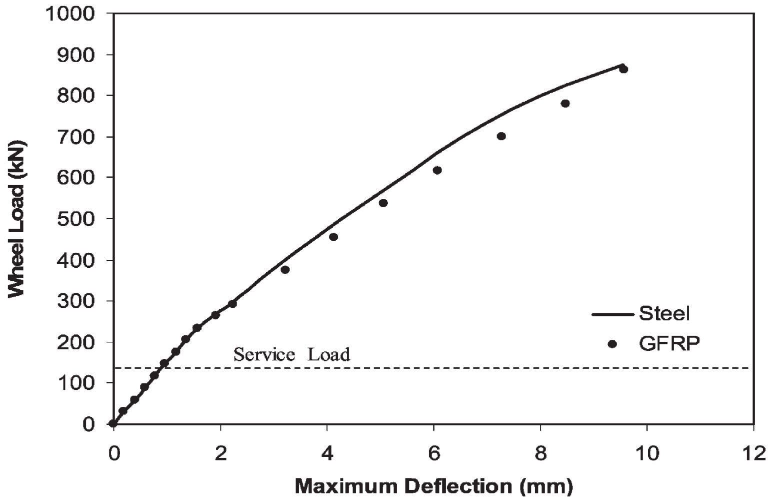


Figure 7 - Comparison between field test data and FEM predictions.

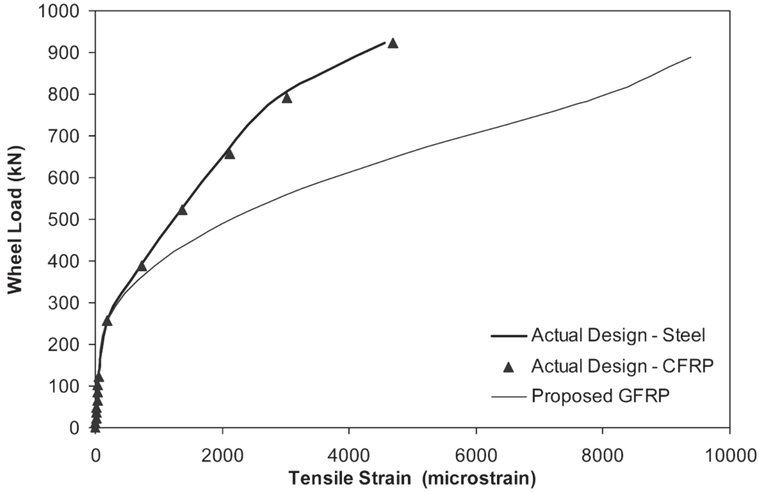


(a) Magog Bridge

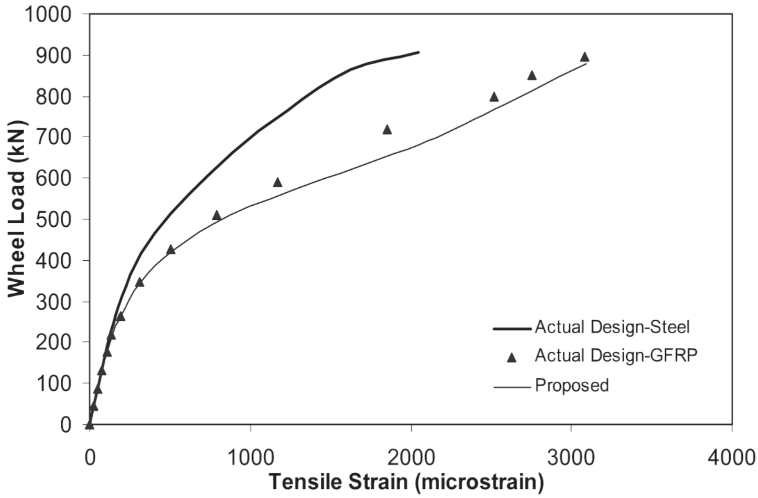


(b) Cookshire-Eaton Bridge

Figure 8 - Behavior of bridge decks under wheel load.

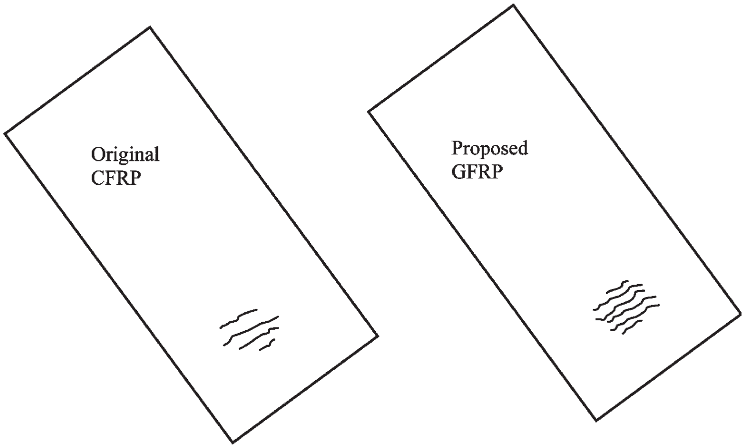


(a) Magog Bridge

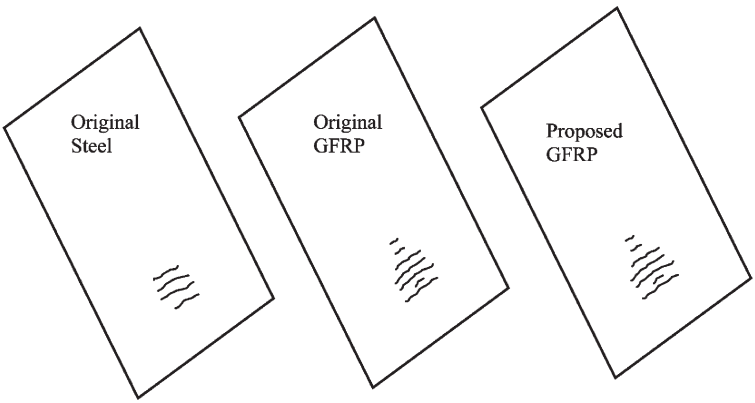


(b) Cookshire-Eaton Bridge

Figure 9 - Comparison between deck slab behavior for actual and proposed GFRP reinforcement ratios.



(a) Magog Bridge



(b) Cookshire-Eaton Bridge

Figure 10 - Comparison between FEM crack patterns at service load level for the proposed reinforcement ratio and the actual deck design.