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Field Testing of Hybrid-Reinforced Concrete Bridge Decks in Quebec

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ABSTRACT: This paper presents the field test results of the hybrid-reinforced Twin Bridges on Chemin de Ste-Chaterine, Sherbrooke, Quebec, Canada. The bridges are hybrid-reinforced slab-on-girder bridges simply supported over a single span of 43.42 m. The concrete deck slab is a 200-mm thick continuous over four spans of 2.65 m each with an average overhang of about 1.00 m on both sides (measured in the perpendicular direction to axis of the girders). The concrete deck slab was reinforced with sand-coated glass fiber-reinforced polymer (GFRP) reinforcing bars in the top mat and with galvanized steel bars in the bottom mat. The bridge is instrumented with fiber-optic sensors (FOS) at critical locations. The bridge was tested for service performance before putting the asphalt layer using three calibrated truckloads to verify the appearance of the flexural cracks. The construction details and the results of the live load field tests are presented. The field tests yielded very small strains in the GFRP reinforcing bars which clarified the arch action effect in the restrained hybrid-reinforced concrete bridge decks.

1 INTRODUCTION

Most of reinforced concrete bridges in Canada are of slab-on-girder type. The deck slabs of these bridges are reinforced with two mats (top and bottom) and connected to the supporting girders through shear connectors (studs). Due to harsh environmental conditions and the excessive use of de-icing salts in winter seasons, the steel reinforced bridge deck slabs exhibit steel corrosion and consequent deterioration. The costs of the repairs and related problems such as delaying/deviating the traffic encouraged the use of the non-corrosive fiber-reinforced polymer (FRP) bars as an alternative reinforcement.

In typical slab-on-girder concrete bridge deck, the top reinforcing mat is closer to the concrete surface and consequently, is susceptible to chloride and chemical exposure which may accelerate the corrosion of steel bars. On the other hand, the bottom reinforcement mat is not susceptible to such exposure. In addition, the design of these bridge decks controls the crack width which limits the penetration of the chloride from the top surface to the bottom reinforcement layer. Thus, design engineers and municipalities proposed the use of the non-corrosive GFRP bars in the top reinforcement mat and maintain the steel bars in the bottom reinforcement mat (hybrid-reinforced concrete bridge deck slabs). This technique is expected to yield cost-effective and durable concrete bridge decks.

Recently, the Ministry of Transportation of Québec (MTQ) initiated an extensive research project in collaboration with the University of Sherbrooke aimed at investigating the structural performance of hybrid-reinforced concrete bridge decks through some new concrete bridges that are being constructed on the extension of Hwy 410 (Sherbrooke, Québec). The first one of these series was the 410 overpass bridge which was constructed and tested in 2010 (Ahmed & Benmokrane 2012).

This paper presents the design criteria and the live load field test of Ste-Catherine overpass Twin Bridges (Sherbrooke, Québec) that were constructed using hybrid reinforcement technique. Once the construction was completed, a live load field testing was conducted on one of them to assess the service performance under calibrated truck wheel loads. The design concept and the results of the first live load field test are discussed herein. The results reported in this paper provide information about the stress levels in the GFRP and steel reinforcing bars obtained from the live load testing as well as the deflection of the supporting steel girders and the distribution factors.

2 BRIDGE DESIGN AND DESCRIPTION

The bridge deck slab was designed according to the flexural design method of the Canadian Highway Bridge Design Code, CHBDC (CSA 2010). The design bending moments were based on a maximum wheel load of 87.5 kN (CL-625 Truck). The design service load for the deck slab was taken as 1.4×0.9×87.5=110.25 kN, where 1.4 is the impact coefficient, and 0.9 is the live load combination factor, while the design factored load was taken as 1.4×1.7×87.5=208.25 kN, where 1.7 is the live load combination factor.

The deck slab was designed based on serviceability and ultimate limit states suing top and bottom clear concrete cover of 50 and 38 mm, respectively. The crack width of the concrete slab and allowable stress limits were the controlling design factors. The MTQ has selected to limit the maximum allowable crack width to 0.5 mm and the stresses in the GFRP bars to 25 and 15% of the ultimate strength of the material under service and sustained loads, respectively. Figure 1 shows the layout of the bridge and Table 1 presents the reinforcement of the bridge deck slabs.

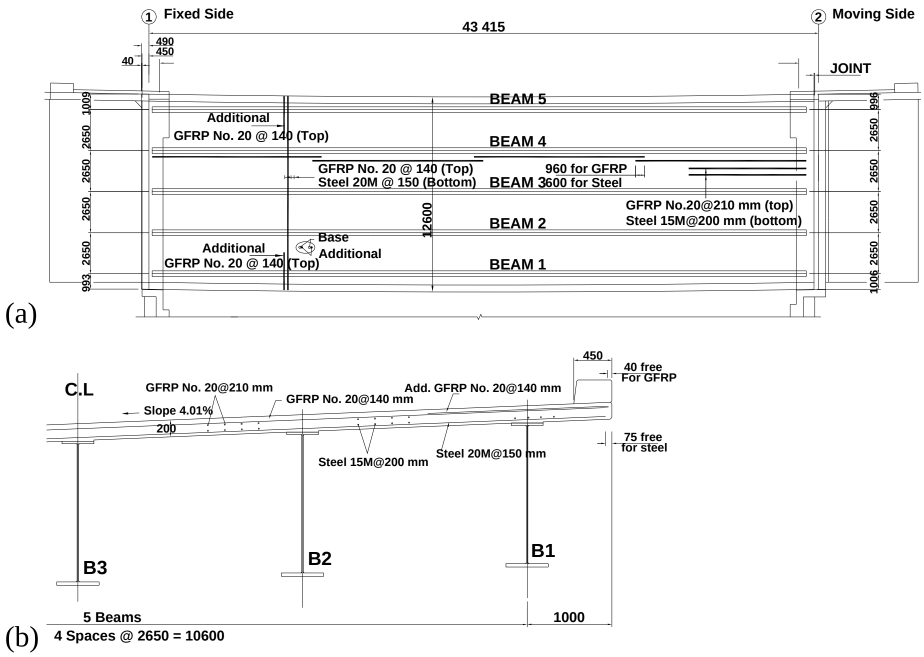


Figure 1. Geometry and reinforcement details: (a) plan view; (b) cross-section.

Table 1. Reinforcement of the bride deck slabs

Element	Bottom reinforcement		Top reinforcement	
	Transverse	Longitudinal	Transverse	Longitudinal
Deck slab	20M Steel @ 150 mm	15M Steel @ 200 mm	No. 20 GFRP @ 140 mm	No. 20 GFRP @ 210 mm
Cantilever	20M Steel @ 150 mm	15M Steel @ 200 mm	No. 20 GFRP @ 70 mm	No. 20 GFRP @ 210 mm

3 INSTRUMENTATION

The bridge was instrumented at critical locations to record the reinforcement strains. Instrumentation was distributed along the mid-span section of the bridge, as shown in Fig. 2. Fiber optic sensors (FOSs) were glued on the transverse GFRP reinforcing bars of the top mat and the transverse steel bars of the bottom mat. The FOS were glued on the GFRP bars (T1 – T5) at locations of support girders (maximum negative moment) and on the steel bars (B1 – B4) at centerlines between the support girders (maximum positive moment) (Fig. 2). The GFRP and steel bars were instrumented at the structural laboratory of the University of Sherbrooke. Thereafter, the bars were shipped to the construction site where they were installed in the designated locations. The objective of using FOS was to allow for the long-term monitoring and future field tests of the bridge. Figure 3 shows the instrumentation of the GFRP and steel bars and reinforcing bars after being installed in the bridge deck slab.

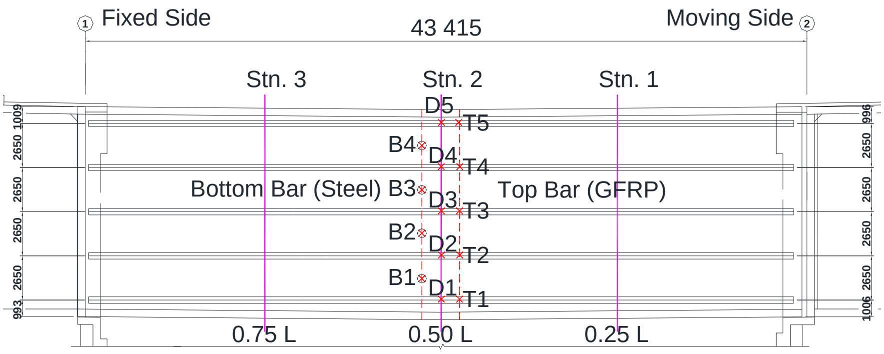


Figure 2: Locations of strain and deflection measurements.

Figure 3: Instrumentation and installation of the reinforcing bars.

During static tests, deflection of the steel girders was measured (D1 – D5) with a theodolite and a system of rulers installed at the bridge midspan (Fig. 2). The deflection was measured also measured using two general purpose digital contact sensors of 50 mm measuring range capacity (GT2-H50) located at D2 and D4 (Beam 2 and Beam 4 – Fig. 2). Figure 4 shows the rulers' system and the two GT2-H50 sensors during the bridge test.

Figure 4: deflection measurements using rulers and GT-H50 sensors.

4 LIVE LOAD TESTING OF BRIDGE

The bridge was tested on October 30, 2012 for service performance as specified by the CHBDC (CSA 2010) using three three-axle calibrated trucks. Trucks No. 1 to 3 had loads of about 72 kN on the front axle and approximately 88 kN on each back axle. Figure 5 shows the complete description of the trucks. The bridge deck was tested using six truck paths using one, two, and three trucks at three different stations (truck stops) in the longitudinal direction of the bridge (Stn 1 – Stn 3). The stations were selected at the quarters and at the middle of the bridge to capture the variation in the straining actions with the trucks location on the bridge. The six paths shown in Fig. 6 were marked on the bridge deck as well as the three stations. Readings were recorded at a truck(s) station when the midpoint of the truck's second and third axle was directly over the station. Figure 7 shows the trucks on the bridge during testing.

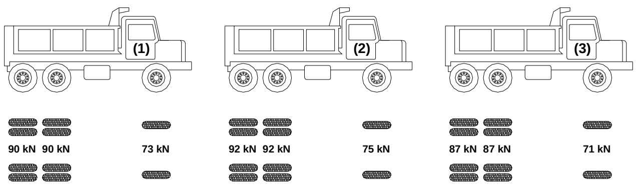


Figure 5: Truck loads

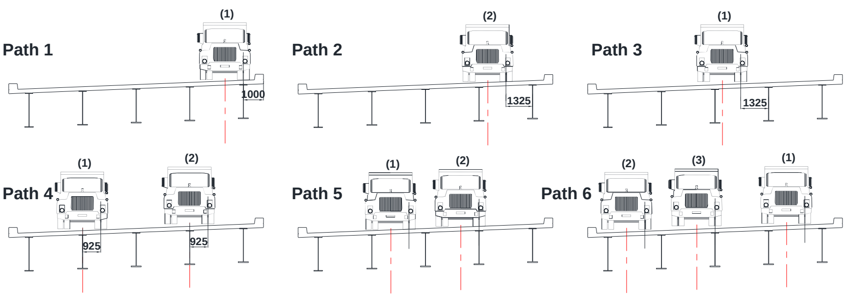


Figure 6: Truck loads and paths of the trucks during testing.

Figure 7: Trucks during testing.

5 LIVE LOAD TEST RESULTS

After each path the bridge deck slab was visually observed for any signs of cracking over the girders (negative moment areas). No cracks were observed at any location in the top surface of the bridge deck at girders locations.

5.1 Strain Measurements

Figure 8 shows the variation of the strain in the bottom transverse steel bars and to the top transverse GFRP bars with the truck locations on the bridge. The maximum strains were recoded when the trucks stopped at the middle of the bridge (Stn 2). Figure 9 presents the maximum strains resulted from the different paths when the trucks were at the middle of the bridge. Generally, the strains were very low in the bottom transverse steel bars and top transverse GFRP bars. The strain in the bottom transverse steel bars at its maximum location was 15 microstrains. Similarly were the strains in the top transverse GFRP bars where its maximum value was 25 microstrains. It should be noted that strains in steel reinforcing bars of the same order as those measured here were reported for similar bridges that were constructed using FRP and steel bars in their concrete deck slabs (El-Salakawy & Benmokrane 2003; El-Salakawy et al. 2005; Ahmed & Benmokrane 2012).

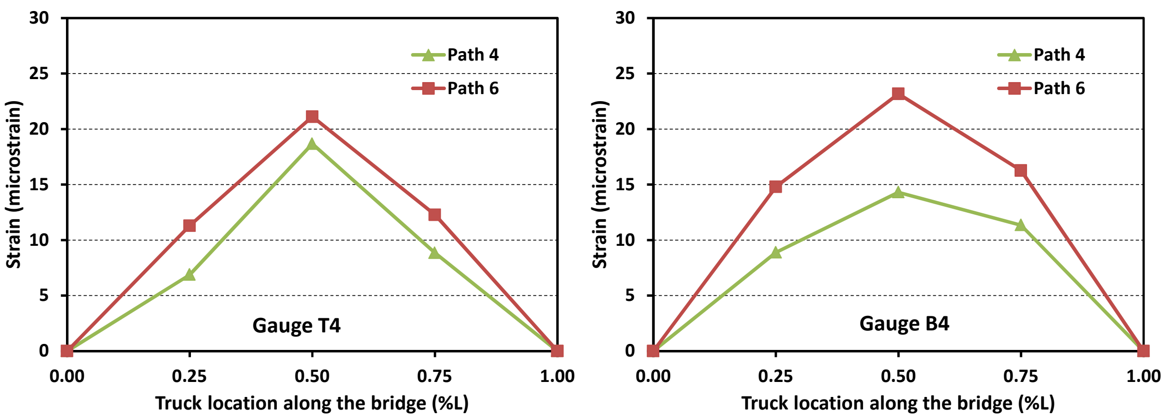
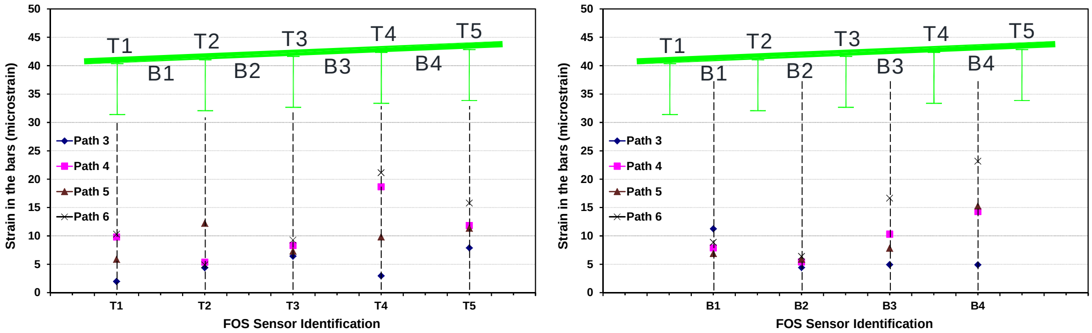


Figure 8: Variation of strains with the truck location.

The maximum measured tensile strains in the GFRP bars are less than 1% of the GFRP's ultimate strains (1333/54200=24600 microstrains). The design service load of 110.25 kN (specified by the CHBDC is greater than the maximum wheel load of the trucks used in the field test which equals 45 kN (Figure 4a) by approximately 2.5 times. However if the maximum values of the strains measured in the field are linearly extrapolated (multiplied by 2.5) the resulting values of the tensile strains will be about 63 microstrains in the top transverse GFRP bars. These values are still less than 1% of the ultimate strains of the GFRP bars. According to the CHBDC (CSA 2010), the allowable stress or strain limit for GFRP bars in concrete slabs is 25% of the material’s ultimate stress or strain values. Similar extrapolation of the strains in the bottom transverse steel bars yielded an strain value of about 2% of the ultimate strain capacity of the steel bars (based on yield strain of 2000 microstrains) which is also very low.



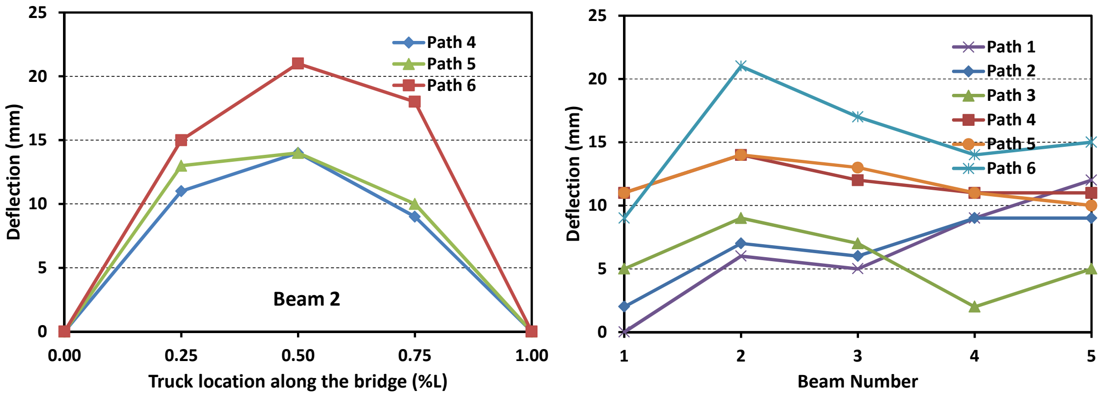
(a) Strains in GFRP bars (b) Strains in steel bars

Figure 9: Reinforcement strains.

The very small measured strains (Fig. 9) indicate that the arch action exists in the restrained hybrid-reinforced concrete bridge decks slabs. Although, the bridge deck was designed according to the flexural design method of the CHBDC (CSA 2010), the slabs did not show a real flexural response due to the arch action effect. Furthermore, since the bridge deck meets the requirements of the CHBDC (CSA 2010) for the empirical method, it was possible to design it using the empirical method. The design of this bridge deck based on the empirical method yields 15M steel bars@300 mm as bottom transverse reinforcement (minimum reinforcement), 15M steel bars@300 mm (minimum reinforcement) as bottom longitudinal reinforcement, and GFRP bars No. 15@300 mm (minimum reinforcement) as top transverse and longitudinal reinforcement. This design saves a significant amount of the transverse reinforcement (steel and GFRP bars) compared to the design using the flexural method.

5.2 Deflection Measurements

Fig. 10a shows the variation of the measured deflection of the steel girders with the trucks location on the bridge while Fig. 10b presents the deflection of the steel girders at the bridge midspan due to trucks located at midspan (Stn 2) for the different paths. The figure indicates that the truck loading was not evenly distributed on the steel girders. The girder closest to the truck path deforms more than those further away. This was more obvious when the truck traveled over or near the edge girder. As shown in Fig. 10b, the single truck following Path 1 (over edge Beam 5-See Fig. 6) produced the peak deflection in Girder 5 of 12.0 mm (L/3617). The peak deflection with the two calibrated trucks traveling simultaneously along Paths 4 and 5 was 14.0 mm (L/3101) in Beam 2. Furthermore, as noted from Paths 4 and 5, when the load is applied onto two lanes the deflection distribution was better and evidenced from Fig. 10b.



(a) Variation with the truck location (b) Deflection of all girders

Figure 10: Maximum Measured deflection of steel girders (trucks at midspan - Stn 2)

The continuous deflection measurements of Beams 2 and 3 using the G2-H50 sensors during the field test are shown in Fig. 11. Comparing the results presented in Figs. 10 and 11 revealed that the conventional rulers and theodolite system yielded very close deflection measurements to that of the G2-H50 sensors. Considering path 6, Stn 2, the deflection of Beam 2 was 20.5 mm from the rulers and theodolite system (Fig. 10b) and 19.51 mm from the G2-H50 sensors (Fig. 11). Thus, this system may be a viable tool when such advanced techniques are not available. The advantage of the advanced optical sensors, however, is the ability to capture the dynamic response when needed.

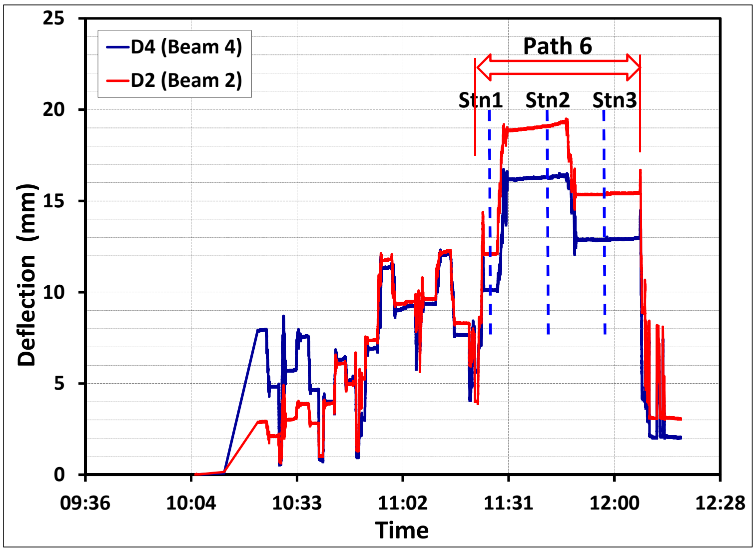


Figure 11: Measured deflection of steel girders using G2-H50 sensors

6 CONCLUSIONS

This paper presents the results of the live load field test of the hybrid-reinforced Ste-Catherine overpass Bridge located on Highway 410 (Sherbrooke, Québec). Based on the details presented herein, the following conclusions can be drawn:

- The maximum tensile strain in the top transverse GFRP bars was less than 1% of the ultimate tensile strain of the GFRP bars. Nevertheless, it is lower than the expected strains using the flexural design method. This result suggests that the CHBDC (CSA 2010) flexural design method overestimates the calculated design moments.
- The very small measured strains in the GFRP reinforcing bars indicate the presence of the arch action between the girders in the restrained hybrid-reinforced bridge decks and ultimately, which in unlikely to occur in field, the failure mode would be by punching shear.
- The conventional rulers and theodolite system may be a viable method to measure the deflection of bridge girders during field tests when the other advanced systems are not available. However, this could not be used if the dynamic response is of interest.

7 ACKNOWLEDGMENTS

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