DESIGN, CONSTRUCTION, AND TESTING OF HYBRID-REINFORCED CONCRETE BRIDGE DECK, 410 OVERPASS BRIDGE

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Abstract

The 410 Overpass Bridge (Sherbrooke, Quebec) bridge is a slab-on-girder type with a skew angle of about 90°. The bridge has nine steel girders simply supported over a single span of 46.60 m. The deck slab is a 225-mm thickness concrete slab continuous over eight spans of 3.50 m each with an average overhang of 1.68 m on one side and 1.50 m on the other side. 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) and vibrating wire strain gauges (VWSG) at critical locations to record internal strain data. Also, the bridge was tested for service performance before putting the asphalt layer using calibrated truckloads to verify the appearance of the flexural cracks in the negative moment areas (over steel girders). 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.

Keywords: Concrete; Deck slab; Bridge; Glass fiber-reinforced polymer (GFRP); Wheel load; Load test; Fiber optic sensors (FOS).

1. Introduction

Since the late 1990s, the Structures Division of the Ministry of Transportation of Quebec (MTQ) has been interested in constructing bridges with an extended service life of 75–150 years. These durable bridges can be constructed by employing the non-corroding FRP reinforcing bars as main reinforcement for the concrete bridge decks. Based on this technique, the MTQ has carried out, in collaboration with the University of Sherbrooke (Sherbrooke, Québec), several research projects utilizing the FRP reinforcement in concrete slabs and bridge barriers. Besides, several demonstration field applications on real bridges [1-4] were carried out using glass FRP bars. Some of these bridges have been in-service for more than 10 years without any signs of deterioration or problems.

Recently, the MTQ started to employ another technique which is the hybrid-reinforced concrete bridge decks. In these bridge decks, only the top mat of the reinforcement is designed using GFRP bars while the bottom mat is maintained as galvanized steel bars. The replacement of the top mat is based on their closer location of the top surface of the slabs. This makes them more susceptible to chloride exposure and consequently accelerated corrosion. On the other hand, the bottom reinforcement mat is not susceptible to such exposure which supports the use of steel bars. This technique is expected to yield cost-effective and durable concrete bridge decks. However, none of these new hybrid-reinforced bridge decks were monitored or tested. Thus, in collaboration with the University of
Sherbrooke, the MTQ aimed at investigating the structural performance of hybrid-reinforced concrete bridge decks through the 410 Overpass Bridge (Sherbrooke, Québec). The design was conducted by the exp engineering firm at Sherbrooke and the Structural Division of the MTQ using the flexural design method of the Canadian Highway Bridge Design Code (CHBDC) [5] which is the commonly used design method for the MTQ.

This paper presents the design criteria and the construction details of the 410 Overpass Bridge (Sherbrooke, Québec) that was constructed using hybrid-reinforcement technique. The design concepts, construction details, and the results of the 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.

2. Description of the Project

The 410 Overpass Bridge is located in Sherbrooke on Highway 410, crosses over Boulevard de l’Université in Québec, Canada. The bridge was constructed using typical slab-on-girder structural system with a skew angle of about 9°. The bridge comprised six lanes, three in each traffic direction, separated using a median barrier of Type MTQ 202ME. A layout of the bridge is shown in Figure 1. The bridge has nine steel girders simply supported over a single span of 46.60 m (Figure 1(a)). The deck slab is a 225-mm thickness (Figure 1(b)) concrete slab continuous over eight spans of 3.50 m each with an average overhang of 1.68 m on one side (east) and 1.50 m on the other side (west). The bridge deck slab was provided with post-tensioned cables spaced at 1000 mm. This is a typical process used by the MTQ for bridge decks with more than 3 traffic lanes for durability purposed. However, it is not included in the design calculations.
3. Design Approaches in the CHBDC CAN/CSA S6-06 [5]

The CHBDC [5] provides the following two methods for the design of concrete bridge decks reinforced with steel and/or FRP reinforcing bars:

3.1 The Empirical Method

As stated by the CHBDC [5] Clause 8.18.4.1, the empirical method is applicable for that portion of the deck slab which is of nearly uniform thickness and bounded by the exterior supporting beams. However, they have to meet the following conditions:
(a) The deck slab is composite with the supporting beams, which are parallel to each other, and the lines of supports for the beams are also parallel to each other,
(b) The ratio of the spacing of the supporting beams to the thickness of the slab is less or equal to 18.0. The spacing of the supporting beams used in calculating this ratio is taken parallel to the direction of the transverse reinforcement,
(c) The spacing of the supporting beams does not exceed 4.0 m and the slab extends sufficiently beyond the external beams to provide full development length for the bottom transverse reinforcement,
(d) Longitudinal reinforcement in the deck slab in the negative moment regions of continuous composite beams is provided for in accordance with Clause 8.19.4 and Section 10, if applicable.

For the empirical design method to apply, a full-depth cast-in-place deck slab should satisfy the conditions specified by Clause 8.18.4.2 (for steel reinforcement) and Clause 16.8.8.1 (for FRP reinforcement) in addition to those of Clause 8.18.4.1.

3.2 The Flexural Method

In the flexural design method, concrete deck slabs shall be analyzed for positive and negative bending moments resulting from loads applied on the slabs. The analysis shall consider the bending moments induced in the longitudinal direction that agree with the assumptions used in the analysis of the transverse bending moments. The cantilever portions of concrete deck slabs shall be analyzed for transverse negative bending moments resulting from loads on the cantilever portions of the slabs or horizontal loads on barriers and railings. The cantilever portions of concrete deck slabs may be analyzed using Clause 5.7.1.6.1 while the deck slabs are analysed using Clause 5.7.1.7.1. The design of sections, however, should be conducted according to Section 8 when steel bars are used and Section 16 when FRP reinforcing bars are used.

It should be noted that the CHBDC [5] states that concrete deck slabs (other than their cantilever portions that are proportioned in accordance with the empirical design method of Clause 8.18.4 for the CL-625 Truck) need not be analyzed for transverse bending moments due to live load. Thus, if the bridge deck slab meets the requirements of the empirical method, it could be designed accordingly.

4. Design of the Concrete Bridge Deck Slab

The bridge deck slab was designed by the MTQ engineers according to the flexural design method of the CHBDC [5]. The applicability of the empirical design method, however, was verified and the bridge deck was designed accordingly to compare the reinforcement amounts resulted from both design methods.
4.1 Material Properties

Normal-strength concrete (Type V MTQ) with a 28-day concrete compressive strength of 35 MPa was used for the bridge deck slab. Sand-coated GFRP bars were used as reinforcement for the top mat of the bridge deck slab (No. 20, 19.1-mm diameter) while galvanized steel bars 20M were used as reinforcement for the bottom mat of the bridge deck slab. The used GFRP bars were manufactured by combining the pultrusion process and an in-line coating process for the outside surface. These bars were made of high strength E-glass fibers with a fiber content of 74% in a vinyl ester resin. The minimum guaranteed tensile strength and the tensile modulus of elasticity of the GFRP bars used in reinforcing the bridge deck slabs were 656 MPa and 47 GPa, respectively.

4.2 Flexural Design Method

As mentioned earlier, the bridge deck slab was designed by the MTQ engineers according to the flexural design method of the CHBDC [5]. 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. 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.

Based on this design approach, the bridge deck slab was entirely reinforced with two reinforcement mats using No. 20 GFRP bars and 20M steel bars. For the bottom reinforcement mat, 20M steel bars spaced at 160 and 200 mm in the transverse and longitudinal directions, respectively. For the top reinforcement mat, No. 20 GFRP bars spaced at 110 and 170 mm in the transverse and longitudinal directions, respectively, were used. Top and bottom clear concrete cover of 60 and 38 mm, respectively, were used. Additional No. 20 GFRP bars spaced at 110 mm were placed in the top transverse layer at the two cantilevers (Figure 1(b)), as well as in the top longitudinal layer at the ends of the deck slab.

4.3 Empirical Design Method

The design of the bridge according to the empirical design method was conducted for comparison with that resulted from the flexural design method. The bridge deck slab satisfies the requirements of the empirical method presented in Section 2.1 of this paper which are:

(a) The bridge deck slab was of uniform thickness and bounded by exterior supporting beams,

(b) The deck slab is composite with the supporting beams, which are parallel to each other, and the lines of supports for the beams are also parallel to each other (Figure 1),

(c) The ratio of the spacing of the supporting beams to the thickness of the slab = (3.5/cosθ)/0.225 = 16.0 < 18.0,

(d) As shown in Figure 1, the spacing of the supporting beams < 4.0 m and the slab extends sufficiently beyond the external beams to provide full development length for the bottom transverse reinforcement as the cantilever length is more than 1/3 the adjacent span.

Thus, this bridge deck might have been designed using the empirical method. The MTQ, however, did not use this method in designing its bridges to date. Considering the steel
reinforcing bars, the area of steel reinforcing bars should be calculated according to Clause 8.18.4.2 whereas the area of FRP reinforcing bars should be calculated according to Clause 16.8.8.1 of the CHBDC [5].

Accordingly, the area of the bottom transverse reinforcing steel bars equals \( \frac{\rho}{\cos^2 \theta} \times d_s \times 1000 \text{ mm}^2/\text{m} \) where \( \rho \) is the reinforcement ratio and equals 0.003, \( \theta \) is the skew angle, \( d_s \) is the effective depth of the deck slab. This yields 15M steel bars @ 300 mm as bottom transverse reinforcement which is the minimum area specified by the CHBDC [5]. The bottom longitudinal reinforcement is set to the minimum reinforcement which is 15M steel bars @ 300 mm. On the other hand, the area of GFRP bars in top longitudinal and transverse direction is equal to \( 0.0035 \times d_s \times 1000 \) which yields GFRP bars No. 15 @ 300 mm (minimum reinforcement). Furthermore, if the deck slab was totally reinforced with GFRP bars, the empirical method would have yielded GFRP Bars No. 20 @ 150 mm \((500d_s/E_{FRP})\) in the bottom transverse directions and No. 15 @ 300 mm in all other directions.

By comparing the designs of the empirical method with that of the flexural method, it could be noticed that the empirical method saves up to 50% of the transverse reinforcement. This could be justified as when the bridge desk slab meets the requirements of the empirical method, the arch action has a significant effect which contributes to reducing the reinforcement amount.

5. Instrumentation and Construction

The bridge was instrumented at critical locations to record internal temperature and strain data. Instrumentation was distributed along the mid-span section of the bridge, as shown in Figure 1(a). Fiber optic sensors (FOSs) were glued on the transverse GFRP reinforcing bars (top mat) at locations of expected maximum stresses (on top of support girders) (T1 to T9). Vibrating wire strain gauges (VWSG) were welded on the transverse steel bars (bottom mat) at the locations of expected maximum stresses (between support girders) (B1 to B8). In addition, two vibrating wire-based concrete gauges (C1 and C2) and two thermometers (Th1 and Th2) were embedded in the concrete to capture the concrete strain and the temperature variation. Figure 1(a) shows the location of the different sensors. 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 and VWSG was to allow for the long-term monitoring and future field tests of the bridge. The FOS and VWSG were controlled and their readings were captured using a 16 Channel DMI unit and CR10X Data logger, respectively.

The construction of the bridge deck slab started on April 2010. The formwork of the bridge deck was completed on April 30, 2010 and the installation of the bottom mat of the steel reinforcing bars started on May 3, 2010 while the installation of the top mat of GFRP reinforcing bars started on May 7, 2010. Figure 2 shows the bridge during the different stages of construction. The placement of the two reinforcement mats was completed on May 14, 2010 and the bridge deck slab was cast on May 17, 2010 and was cured for 2 weeks. The approach slabs of the bridge were cast on May 31, 2010 and the barrier walls were cast on June 9, 2010. As one of the main objectives of this project was to verify the flexural cracking that might occur during the field testing, the bridge was tested before pavements. The live load field test was conducted on Saturday, June 19, 2010. On June 25, 2010, the bridge deck was paved with a 65-mm-thick layer of asphalt. The bridge was opened for traffic at the end of October 2010.
6. Live Load Testing of Bridge

The bridge was tested on June 19, 2010 for service performance as specified by the CHBDC [5] using three three-axle calibrated trucks. Truck Nos. 1 to 3 had loads of about 72 kN on the front axle and approximately 88 kN on each back axle. The three trucks were used simultaneously to carry out the field test (static). During testing, the 16-channel FOS data-acquisition system and the CR10X data logger were used to collect the FOS and VWSG readings.

Five paths (1, 2, 3, 4, and 5), which were predicted to give the maximum moments in the deck slab, were identified and marked on the bridge (Figure 3(a)). There were no paths over the cantilevers because the formworks of the two cantilevers of the bridge were yet to be removed on the day of testing. Thus, it was decided to keep the paths over the interior spans and exclude the cantilevers. Two stations (truck stops) were also marked on the longitudinal direction of the bridge at selected positions. One at the quarter of the bridge deck and the second one at the middle of the bridge however the second one is reported herein because it matches the sensors’ locations. Readings were recorded at a truck station when the midpoint of the truck’s second and third axle was directly over the station. Figure 3(b) shows the trucks on the bridge during testing.
7. 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.

Figure 4 shows the strain measurements from the VWSG attached to the bottom transverse steel bars and the FOS attached to the top transverse GFRP bars resulted from the different paths. Generally, the strains were very low in the bottom transverse steel bars and top transverse GFRP bars. The strains in the bottom transverse steel bars at its maximum location did not exceed 70 microstrains. Similarly were the strains in the top transverse GFRP bars where its maximum value was less than 50 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 [2, 6].

These maximum measured tensile strains in the GFRP bars are less than 1% of the GFRP’s ultimate strains (13,967 microstrain). 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 44 kN by approximately 2.51 times. However if the maximum values of the strains measured in the field are linearly extrapolated (multiplied by 2.51) the resulting values of the tensile strains will be about 125 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 [5], 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 9% of the ultimate strain capacity of the steel bars (based on yield strain of 2000 microstrain) which is also very low. The concrete strain measured at the level of the top reinforcement layer due to the live load test was -29 microstrains which is also very low.

![Figure 4. Strain measurements using FOS and VWSG.](image)

Figure 4 shows that the measured strains in GFRP reinforcing bars resulted from the field test are very small. This indicates that the arch action exists in the restrained hybrid-reinforced concrete bridge decks slabs. However, the bridge deck was designed according to the flexural design method of the CHBDC [5], 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 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 more than 50% of the transverse reinforcement (steel and GFRP bars) compared to the design using the flexural method.

8. Conclusions
This paper presents the construction details and the test results of the 410 Overpass Bridge located on Highway 410 (Sherbrooke, Québec). Based on the details presented herein and the results of the field-loading test, the following conclusions can be drawn:

- During the live load test, 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 S6-06 2006) 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 in the restrained hybrid-reinforced bridge decks. When the bridge decks meet the requirement of the CHBDC [7] concerning the empirical method, it could be design accordingly which could save up to 50% of the transverse reinforcement.

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10. References