Fibre reinforced polymer composite bars for the structural concrete slabs of a Public Works and Government Services Canada parking garage

Brahim Benmokrane, Ehab El-Salakawy, Zoubir Cherrak, and Allan Wiseman

Abstract: Public Works and Government Services Canada (PWGSC) is currently undertaking the reconstruction of the interior structural slabs of the Laurier–Taché parking garage (Hull, Quebec). An agreement between PWGSC and the Université de Sherbrooke was reached to use fibre reinforced polymer (FRP) reinforcement technology in some demonstration areas of the Laurier–Taché parking garage. Based on this agreement, three different designs using carbon and glass FRP bars were carried out in collaboration with the project consultant. These designs were made according to CAN/CSA-S806-02, Design and Construction of Building Components with Fibre-Reinforced-Polymers. To verify the design, it was agreed to conduct laboratory testing on full-scale slab prototypes of the proposed designs. This paper presents the results of the experimental tests where six full-sized one-way slabs were constructed and tested. The slabs were 3100 mm long × 1000 mm wide × 150 mm deep and were tested under four-point bending. The test results along with a comparison to the predictions of CAN/CSA-S806-02 Code and ACI 440.1R-03 design guidelines are presented. Recommendations for design of FRP-reinforced concrete slabs are also given.

Key words: concrete slabs, parking garages, FRP bars, flexural behaviour, cracking, deflection.

Introduction

Fibre reinforced polymer (FRP) rods are used as reinforcement for concrete structures in which the corrosion of steel reinforcement has typically led to significant deterioration and rehabilitation needs. The noncorrosive nature of the FRP rods is beneficial for improved durability. Thus, Public Works and Government Services Canada (PWGSC) decided to use the FRP technology in the reconstruction of the interior structural slabs of the Laurier–Taché parking garage. However, the relatively low modulus of FRP composites, especially glass FRP (GFRP), compared with steel reduces the...
serviceability performance of the flexural members. Having the same ultimate capacity, FRP-reinforced members will have larger deflections and crack widths than steel-reinforced members. Accordingly, in most cases, serviceability requirements govern the design of FRP-reinforced concrete members (Matthys and Taerwe 1995; Michaluk et al. 1998; Alkhrdaji et al. 2000; El-Salakawy and Benmokrane 2003).

Several codes and design guidelines for concrete structures reinforced with FRP composite bars have been recently published: CAN/CSA-S6-00 (CSA 2000), ISIS-M03-01 (ISIS Canada 2001), ACI 440.1R-03 (ACI 2003), including the new Canadian code, Design and Construction of Building Components with Fibre Reinforced Polymers (CAN/CSA-S806-02 (CAS 2002)). Based on these codes and design guidelines, several field applications, especially concrete bridges, have been recently carried out in North America using FRP composite bars as reinforcement for the concrete deck slabs (Steffen et al. 2001; El-Salakawy and Benmokrane 2003; Benmokrane et al. 2003). In the constructed bridges, different FRP reinforcement types, ratios, and configurations were used based on flexural behaviour of the concrete deck slab and different serviceability criteria. However, to date, no field applications for building and especially parking structures have been carried out.

This paper investigates the flexural behaviour and serviceability performance of one-way concrete slabs reinforced with FRP composite bars (glass and carbon) and premanufactured carbon FRP grids (CFRP), which were designed to replace the interior structural slabs of the Laurier–Taché parking garage (Hull, Quebec).

### Laurier–Taché parking garage

The Laurier–Taché parking garage, located in Hull (Quebec), consists of three wings, Maisonneuve Wing, Laurier–Taché East Wing, and Laurier–Taché West Wing, as shown in Fig. 1. The two newest wings have three levels, while Maisonneuve Wing has only two levels. This garage, which was originally constructed in the early 1970s, is a skeletal building made of reinforced concrete columns, beams, and slabs. As a result of corrosion of the steel reinforcement, the structural concrete slabs of this parking garage suffered from the partial or total loss of steel reinforcement and spalling of concrete cover at several locations, as shown in Fig. 2. The PWGSC decided to rehabilitate this parking garage by replacing all structural slabs without damaging the other structural elements (beams and columns). In addition, some damaged beams and columns will be repaired. This reconstruction is being carried out in three phases over a period of 3 years. Phase I includes Maisonneuve Wing (2001–2003) and is scheduled to commence in the spring of 2004. The Université de Sherbrooke in collaboration with the PWGSC design team established a location within the garage to use the FRP composites as embedded reinforcement for the reconstruction of some designated slabs located in the first floor level on Laurier–Taché East Wing. The work includes the design of structural concrete slabs reinforced with FRP bars, preparation of specifications and construction details, laboratory testing of full-scale slabs of the proposed designs, instrumentation, and monitoring. The objectives of this project are:

- to build more durable and maintenance-free concrete structures. Building more durable long-lasting structures will advance the commitment of the Canadian Federal Government to sustainability and reduction of green house gas emission
- to implement FRP reinforcing bar technology in parking garages
- to evaluate long-term performance of FRP under service and environmental conditions
- to compare the in-service behaviour of FRP with that of steel
- to validate and (or) improve the design codes and guidelines
- to enhance the confidence of using FRP as concrete reinforcement based on the real-life monitoring information

Based on the pouring sequence suggested by the project consultant, different types of FRP composites are to be used to reinforce the slabs between axis LT18 to LT28 and LTB to LTC (about 1000 m²) as shown in Fig. 3. The slabs between axis LT28 to LT30 and LTA to LTB (about 160 m²) are to be reinforced with conventional steel as a control area. It should be mentioned that based on the construction details and the field test results of the FRP-reinforced concrete slabs, other areas of the parking garage may be reinforced with FRP bars.

Three different designs were proposed by the research personnel at the Université de Sherbrooke and approved by the project consultant. These designs were made according to CAN/CSA-S413-94 (CSA 1997) for parking structures and CAN/CSA-S806-02 (CSA 2002) for design and construction of building components with fibre reinforced polymers that include:

- glass FRP bars (both top and bottom mats)
- carbon FRP bars (both top and bottom mats)
- glass FRP bars for top mat and carbon FRP bars for bottom mat

In addition, the proposed designs took into account the concerns raised by the project design team regarding the fire rating of 2 h as specified by CAN/CSA-S413-94. On page 169 of the CSA S806-02 Code, it is stated that “... the fire resistance of FRP-reinforced concrete slabs depends on the critical temperature of FRP reinforcement, the thickness of the concrete cover, and the type of aggregate in the concrete mix. The critical temperature is defined as the temperature at which the reinforcement loses enough of its strength (typically 50%) that the applied load can no longer be supported.” Based on Figures T1 to T8 provided by the Code, for 2-h fire rating, approximately 80 mm of concrete cover is needed (Figure T6, page 175) for the 150-mm thick slab. In case of a fire from one side of the slab, and provided that the concrete cover on that side of the slab is only 30 mm, the FRP reinforcement on that side will be completely lost before 2 h. However, applying the aforementioned concept and assuming that the FRP reinforcement on the other side, away from the fire, will be capable, by itself, of carrying the whole applied load for more than 2 h, establishes a safe design of the slab. As the concrete cover for the FRP main reinforcement on the other side of the fire is at least 101 mm,
which is greater than the required 80 mm, the slab will have more than 2-h fire rating.

The design calculations using carbon FRP bars and taking into account the fire rating of 2 h are given in Appendix A. In Appendix A, in the case of fire on the top of the slab, it is assumed that the top reinforcement mat will be totally lost and no moment can be transferred across the supporting beams. Consequently the slab panel will be simply supported on the beams over a span of 4.0 m, which will result in a bending moment of 26.58 kN·m in the bottom layer. However, in the case of fire on the bottom side of the slab, it is assumed that the bottom reinforcement mat will be totally lost and a major crack will be developed at the middle of the slab panel. Consequently the slab panel will be divided into two parts. Each is a cantilever of 2.0-m length hanging from the two adjacent panels, which resulted in a bending moment of 43.08 kN·m in the top layer. These developed moments can be safely carried by a single layer of FRP reinforcement.

Figure 4 shows the carbon FRP reinforcement (carbon FRP bar No. 10) details in the Laurier–Taché parking garage. The design calculations using glass FRP bars (top and bottom mats), and glass FRP bars (top mat) and carbon FRP bars (bottom mat), can be found elsewhere (El-Salakawy et al. 2003b).

Laboratory experimental program

Material properties

The slabs were constructed using normal-weight ready-mixed concrete with an average targeted concrete compressive strength of 35 MPa similar to the one that will be used in the field. However, the obtained 28-d compressive strength, from compressive tests that were performed on three 150 mm × 300 mm cylinders from each concrete batch, varied between 37 and 40 MPa. The properties of the CFRP and GFRP bars and grids as well as steel bars that were used in reinforcing the slabs are listed in Table 1. Figure 5 shows a photograph of the reinforcing bars used in the slabs.

Test prototypes

A total of six full-sized one-way slab prototypes were constructed and tested to failure. The slabs were 3100 mm long × 1000 mm wide × 150 mm deep. The six slab prototypes included three slabs reinforced with carbon and glass FRP bars (S-CCB, S-GGB, and S-CGB), one slab with carbon FRP rods (S-CCR), one with premanufactured carbon FRP grids (S-CCG), and one control slab reinforced with conventional steel bars. Five of these slabs were identical to the proposed design of the structural slabs of the Laurier–
Taché parking garage, whereas the remaining slab (S-CCR) represents a repair work that was done a few years ago to some of the structural slabs of the same parking structure.

The designations of the slabs can be explained as follows. The first character, S, denotes the slab and the last character denotes the type of FRP bar, B for bar, G for grid, and R for rod. The second and third characters denote the type of FRP fibre, C for carbon and G for glass in the bottom and top mats, respectively. Table 2 gives full details of the reinforcement configuration. Also, Fig. 6 shows the dimensions and reinforcement details of the slab prototypes.

The balanced reinforcement ratio, $\rho_{fb}$, of a concrete section is the reinforcement ratio at which a simultaneous rupture of FRP bars (yielding for steel) and crushing of concrete occur. The FRP-reinforced test slabs were designed such that the actual reinforcement ratio is equal to or greater than the balanced reinforcement ratio, $\rho_{fb}$, which is given in Section 8.2.1 of ACI 440.1R-03 as

$$\rho_{fb} = 0.85 \beta_1 \frac{f_t}{f_{tu}} \frac{E_t \varepsilon_{cu}}{f_{tu} E_i \varepsilon_{cu} + f_{tu}}$$

[1]
In the Canadian code CAN/CSA-S806-02 (Clause 8.2.1), it is specified that the FRP-reinforced concrete section shall be over-reinforced ($\rho_f \geq \rho_{rh}$). As it can be seen in Table 2, all FRP-reinforced slabs satisfy this condition except slab S-CCG, reinforced with carbon grids, which was under-reinforced. However, according to the same code (CAN/CSA S806-02, Clause 5.1.2), the reinforcement configuration of slab S-CCG can be accepted if proven by experiment that it satisfies both serviceability and strength requirements.
Instrumentation

Electrical resistance strain gauges were used to measure tensile strains in the reinforcing bars and compressive strains in the concrete. These gauges were glued on several locations across the slab width at the midspan section. The midspan deflection was measured using two linearly varying displacement transformers (LVDTs) fastened at each side of the slab. Two high-accuracy LVDTs (0.001 mm) were also installed at positions of first cracks to measure crack widths. A data acquisition module monitored by a computer was used to record the readings of the strain gauges, LVDTs, and the load cells.

Test set-up and procedure

The slabs were tested under four-point bending over a simply supported clear span of 2500 mm and a shear span of 1000 mm, as shown in Fig. 7. The load was monotonically applied at a stroke-controlled rate of 2.0 mm/min to achieve failure in 30–40 min. The loading was stopped when the first two cracks appeared and the initial crack widths were measured with the aid of a hand-held 50x microscope, then the two high-accuracy LVDTs were installed to measure crack width electronically. During loading, the formation of cracks on the sides of the slabs were also marked and recorded.

It should be noted that the slab prototypes were tested over a clear simple span of 2.5 m while the actual structural slabs of Laurier–Taché parking garage are continuously supported over an effective span of 3.6 m. The continuity of the slabs results in redistribution of the top and bottom moments, especially at high loads. However, this will not affect the flexural behaviour comparison to actual slabs, except for deflection values, as all test results are analyzed based on the applied moment (section analysis) at relatively low load levels compared with the ultimate capacity of the slabs. The validation of comparison to deflection values (based on member analysis) will be discussed in details in the following section.

Test results

Test results will be presented in terms of deflection, cracking, strains in reinforcing bars and concrete, ultimate capacity, deformability, and mode of failure. The test results are analyzed and compared at the two design load levels, service (14.5 kN·m) and ultimate (20.2 kN·m), according to CAN/CSA-S413-94 Parking Structures (CSA 1997) as shown in Appendix A. The test results presented are consistent with previous work that was done by the authors and other researchers (El-Salakawy et al. 2003a; El-Salakawy and Benmokrane 2003; Michaluk et al. 1998).

Deflection characteristics

Figure 8 shows the midspan deflection versus applied moments for the tested slabs. For the control steel-reinforced slab (S-ST), the load-deflection curve was trilinear with yielding plateau. For FRP-reinforced slabs, the load-deflection curve was bilinear. The first part up to the cracking moment \( M_{cr} = 9.5 \) to 10.3 kN·m — the moment due to the own weight of the slab, 2.7 kN·m, was not included) was similar to the control slab representing the behaviour of the uncracked slab utilizing the gross inertia of the concrete cross section, while the second part represents the cracked slab with reduced inertia.

The maximum midspan deflection in an elastic member can be expressed as (Wang and Salmon 1985)

\[
\delta = \beta_a \frac{M_{cr} I^2}{E_c I_c}
\]

where \( \beta_a \) is a coefficient that depends on the degree of fixity at supports, the variation of the moment of inertia along the span, and the distribution of loading.

For the case of a continuous one-way slab with more than three equal spans under uniformly distributed loads, the
Table 1. Properties of reinforcing bars

<table>
<thead>
<tr>
<th>Reinforcement type</th>
<th>Bar No.</th>
<th>Diameter (mm)</th>
<th>Area (mm²)</th>
<th>Modulus of elasticity (GPa)</th>
<th>Tensile strength (MPa)</th>
<th>Ultimate strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bars</td>
<td>CFRP</td>
<td>10</td>
<td>9.50</td>
<td>71</td>
<td>114</td>
<td>1535</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>9.50</td>
<td>71</td>
<td>45</td>
<td>775</td>
</tr>
<tr>
<td></td>
<td>GFRP</td>
<td>16</td>
<td>15.90</td>
<td>198</td>
<td>45</td>
<td>755</td>
</tr>
<tr>
<td>Carbon</td>
<td>FRP rods</td>
<td>8</td>
<td>8.00</td>
<td>49</td>
<td>155</td>
<td>2800</td>
</tr>
<tr>
<td>Carbon</td>
<td>FRP grids</td>
<td>C10</td>
<td>8 × 4.9</td>
<td>39.2</td>
<td>100</td>
<td>1200</td>
</tr>
<tr>
<td>Steel</td>
<td>10M</td>
<td>11.30</td>
<td>100</td>
<td>200</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2. Details of slab reinforcement.

<table>
<thead>
<tr>
<th>Slab</th>
<th>ρact (%)</th>
<th>ρact/ρth</th>
<th>Strong direction Top</th>
<th>Bottom</th>
<th>Weak direction Top</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-CCB</td>
<td>0.49</td>
<td>1.49</td>
<td>No. 10@125 carbon bar</td>
<td>No. 10@125 carbon bar</td>
<td>No. 10@300 carbon bar</td>
<td>No. 10@300 carbon bar</td>
</tr>
<tr>
<td>S-GGB</td>
<td>1.77</td>
<td>3.2</td>
<td>No. 16@100 glass bar</td>
<td>No. 16@100 glass bar</td>
<td>No. 10@200 glass bar</td>
<td>No. 10@200 glass bar</td>
</tr>
<tr>
<td>S-CGB</td>
<td>0.49</td>
<td>1.49</td>
<td>No. 16@100 glass bar</td>
<td>No. 10@125 carbon bar</td>
<td>No. 10@200 glass bar</td>
<td>No. 10@200 glass bar</td>
</tr>
<tr>
<td>S-CCR</td>
<td>0.42</td>
<td>2.53</td>
<td>No. 8@125 carbon rod</td>
<td>No. 8@125 carbon rod</td>
<td>No. 8@300 carbon rod</td>
<td>No. 8@300 carbon rod</td>
</tr>
<tr>
<td>S-CCG</td>
<td>0.34</td>
<td>0.69</td>
<td>(C10) 100 × 200 mm carbon grids</td>
<td>(C10) 100 × 200 mm carbon grids</td>
<td>(C10) 100 × 200 mm carbon grids</td>
<td>(C10) 100 × 200 mm carbon grids</td>
</tr>
<tr>
<td>S-ST</td>
<td>0.9</td>
<td>0.2</td>
<td>No. 10M@150 steel</td>
<td>No. 10M@150 steel</td>
<td>No. 10M@150 steel</td>
<td>No. 10M@150 steel</td>
</tr>
</tbody>
</table>

Fig. 5. Fibre reinforced polymer and steel reinforcement used in the tested slabs.
Fig. 6. Test specimens: (a) Slab S-GGB (Configuration 1), (b) Slab S-CCB (Configuration 2), (c) Slab S-CGB (Configuration 3), (d) Slab S-CCR (Configuration 4), (e) Slab S-CCG (Configuration 5), and (f) Slab S-ST (Configuration 6).
maximum deflection is expected to be in the first panel (span) with a zero moment at the end support. In this case, the value of $\beta_a$ is 1/12.16 (using the values of $M^{\text{ve}} = 0.077w l^2$ and $M^{-\text{ve}} = 0.107w l^2$ (CPCA 1995)). Thus, the maximum deflection of the actual slabs (span of 3.6 m) in the parking structure can be given by

$$\delta_{\text{field}} = \frac{1}{12.16} \frac{M_{\text{ser}} (3.6)^2}{E (l_c)}$$

For the case of a simply supported one-way slab under two equal concentrated loads applying at a shear span of 0.4l (test set-up), the value of $\beta_a$ is 1/10.17. Thus, for the same value of applied moment, $M_{\text{ser}}$, the maximum deflection of the tested slabs (span of 2.5 m) in the laboratory can be given by

$$\delta_{\text{exp}} = \frac{1}{10.17} \frac{M_{\text{ser}} (2.5)^2}{E (l_c)}$$

and

$$\delta_{\text{exp}} = \frac{12.16 \times (2.5)^2}{10.17 \times (3.6)^2} \delta_{\text{field}} = 0.576 \delta_{\text{field}}$$

At service load level, the allowable deflection limit by Canadian Code (CAN/CSA-S806-02) should not exceed

$$\delta_{\text{field}} \leq \frac{\text{span}}{360} = \frac{3600}{360} = 10 \text{ mm}$$

Table 3. Summary of the test results.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Failure moment (kN-m)</th>
<th>Deflection (mm)</th>
<th>Strain in reinforcement (microstrain)</th>
<th>Strain in concrete (microstrain)</th>
<th>Deformability factor</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Service</td>
<td>Failure</td>
<td>Service</td>
<td>Failure</td>
<td>Service</td>
<td>Failure</td>
</tr>
<tr>
<td>S-CCB</td>
<td>75</td>
<td>3.51</td>
<td>56.1</td>
<td>1082</td>
<td>10400</td>
<td>486</td>
</tr>
<tr>
<td>S-GGB</td>
<td>91</td>
<td>3.11</td>
<td>65.8</td>
<td>1020</td>
<td>11955</td>
<td>425</td>
</tr>
<tr>
<td>S-CGB</td>
<td>76</td>
<td>3.39</td>
<td>57.0</td>
<td>1028</td>
<td>10860</td>
<td>522</td>
</tr>
<tr>
<td>S-CCR</td>
<td>85</td>
<td>3.38</td>
<td>60.0</td>
<td>998</td>
<td>10430</td>
<td>648</td>
</tr>
<tr>
<td>S-CCG</td>
<td>67</td>
<td>4.70</td>
<td>79.5</td>
<td>1262</td>
<td>14700</td>
<td>665</td>
</tr>
<tr>
<td>S-ST</td>
<td>55</td>
<td>1.90</td>
<td>57.6</td>
<td>611</td>
<td>11230</td>
<td>249</td>
</tr>
</tbody>
</table>

\[ \delta_{\text{exp}} \leq 0.576 \times 10.0 = 5.76 \text{ mm} \]

Thus, the allowable deflection for the laboratory-tested slabs (simply supported span of 2.5 m) is 5.76 mm. As listed in Table 3, at service load level (14.2 kN-m), the measured deflection for the tested slabs varied between 1.90 mm (S-ST) and 4.7 mm (S-CCG), which is less than the allowable code limits.

For the sake of comparison, at service load level, the ratio of the measured deflection for slabs S-CCB, S-GGB, S-CGB, S-CCR, and S-CCG reinforced with FRP composites to that of the control slab, S-ST, reinforced with steel were 1.82, 1.65, 1.78, 1.78, and 2.47, respectively.

Measurements that the FRP-reinforced slabs would give before failure. This will be further explained in the following section of this paper dealing with deformability.

Cracking

For all slabs crack formation was initiated at a moment, \( M_{cr} \), of 12 to 13 kN-m. The experimental values of the cracking moments, listed in Table 4, included the moment due to the own weight of the slab (2.7 kN-m).

Cracks in the pure bending zone (the middle 500 mm) were vertical cracks perpendicular to the direction of the maximum principal stress induced by pure moment. Cracks outside the pure bending zone (shear span of 1000 mm on each side) started similarly to flexural cracks, but as the load was increased shear stress become more dominant and induced inclined cracks. Figures 9 and 10 show schematic drawings of the crack patterns for the tested slabs at two load stages: service and failure.

It can be noted that at service load level the number of cracks developed in FRP-reinforced slabs was in the range of 6 (slab S-GGB) to 8 (slab S-CCG), while this number was in the range of 12 (slab S-GGB) to 16 (slab S-CGB) at failure. In addition, the crack penetration depth and spacing at service load level were in the range of 40–80 and 175–120 mm (S-GGB and S-CGB), respectively. These values were 110–130 and 65–43 mm (S-GGB and S-CCG), respectively, at failure (see Table 4). Figure 11 shows the final crack patterns of the tested slabs at failure. In other words, the ratios of number of cracks, crack penetration depth, and crack spacing at failure to those at service load level were in the range of 2, 1.5 to 2, and 0.3 to 0.4, respectively. This is also a good indication of the ample warning that the FRP-reinforced slabs would give before failure.

For crack width measurements, Fig. 12 shows the variation of the maximum measured crack width against the applied moment for the tested slabs. For slabs reinforced with FRP bars, the crack width varies linearly with the load up to failure. The measured crack widths ranged between 0.1 (S-ST) and 0.34 mm (S-CCG), which is less than the allowable code limits of 0.5 mm (ACI 2003).

For cracking control, the new Canadian Code (CAN/CSA-S806-02) introduced a parameter, \( z \)

\[ z = \frac{E_s k_b f_{1}^{3/2} d}{E_t} \]

The parameter \( z \) should not exceed 45 000 N/mm for interior exposure and 38 000 N/mm for exterior exposure when FRP reinforcement is used. Table 4 gives the \( z \) values for the tested slabs based on the actual measured strains in the reinforcing bars at service load level of 14.2 kN-m. These \( z \) values ranged between 8 775 (S-ST) and 18 589 (S-CCG), which are below the allowable code limit of 38 000 N/mm.

Strains in reinforcement and concrete

Figure 13 shows the measured midspan strains in reinforcement as well as in concrete versus the applied moment. For the five slabs reinforced with FRP bars, it can be noted that the strains vary linearly with the increased load up to failure and the maximum measured strains (10 400 to 11 955 microstrain) were less than the ultimate strains of the FRP materials except for slab S-CCG (reinforced with carbon FRP grid), which failed by rupture of FRP grids (14 700 microstrain). This was expected, since slab S-CCG was under-reinforced. The maximum measured compressive strains in concrete at failure ranged between 3230 and 3710 microstrain for all tested slabs.

As listed in Table 3, at service load level of 14.2 kN-m, the maximum measured strains in FRP bars varied between 998 (S-CBB) and 1262 microstrain (S-CCG), which are less than 2000 microstrain.

According to ISIS Canada Design Manual (ISIS Canada 2001), if the measured strain in FRP reinforcement at service load level is below 2000 microstrain, it means that the concrete member would satisfy the serviceability limits.

Furthermore, creep-rupture is a well-known phenomenon for FRP material that is addressed by FRP design codes and guidelines (CAN/CSA S806-02, ACI 440.1R-03) through limiting the stress level in FRP bars under sustained loads.
According to ACI 440.1R-03, these stress limits depend on the type of fibre used and are 0.2, 0.3, and 0.55 of $f_u$ for glass, aramid, and carbon FRP, respectively, under unfactored sustained loads. However, in CAN/CSA S806-02, these stress limits were defined only for glass FRP bars ($0.3f_u$ under factored sustained loads). In the case of the glass FRP-reinforced slab, the calculated stress due to the factored sustained load of 8.97 kN·m (the sustained load is defined by ACI 440.1R-03 as dead load + 20% of the live load) is approximately 118 MPa ($0.156f_u$) in the glass FRP bars, which is well below the code allowable limits of $0.3f_u$ (CAN/CSA S806-02). The experimentally measured strain for the slab reinforced with glass FRP bars, S-GGB, at a load level equal to the sustained load ($M_{mi} = 8.97$ kN·m) was 655 microstrain, as shown in Fig. 13. This strain would cause a stress equal to 72.1 MPa in the GFRP bars, which is even less than the calculated value.

For the control slabs reinforced with steel (S-ST), a typical steel-yielding plateau was obtained with a maximum measured strain of approximately 11 230 microstrain. After steel yielding, the compression strains in concrete increased (3500 microstrain), resulting in failure by concrete crushing.

**Ultimate capacity and mode of failure**

Four slabs reinforced with FRP bars failed by the crushing of concrete in compression, whereas the control steel-reinforced slab S-ST and the slab reinforced with carbon grids S-CCG failed by steel yielding and FRP rupture, respectively, followed by crushing of concrete. These results were expected because the two slabs S-ST and S-CCG were
Fig. 11. Photograph of the final crack patterns at failure.
under-reinforced and the remaining four slabs were over-reinforced. In addition, all slabs reinforced with FRP reinforcement showed an increase of the ultimate carrying capacity of 22–65% compared with the control slab, S-ST. Failure moments for the tested slabs are listed in Table 3.

Deformability

Ductility of a reinforced concrete element provides a measure of the energy absorption capability. Ductility of concrete members reinforced with steel bars is defined as the ratio of deflection or curvature values at ultimate to those at yielding of steel. As there is no yielding point for FRP composite bars, a parameter for comparing the ductility behaviour of FRP-reinforced beams with that of steel-reinforced ones has been developed by Jaeger et al. (1995); it is referred to as the $J$-factor or deformability factor. This factor can be defined as the ratio of energy absorption (area under the moment–curvature curve) at ultimate strength level to the energy absorption at service level. In other words, it is calculated as the product of the ratio of the moment at ultimate, $M_{ult}$, to the moment at a certain service condition, $M_c$, called the strength factor, and the ratio of the curvature at ultimate, $\psi_{ult}$, to curvature at the same service condition, $\psi_c$, called the curvature factor.

$$ J = \frac{M_{ult} \psi_{ult}}{M_c \psi_c} \quad \text{(9)} $$

The service condition is defined as the upper limit of elastic behaviour of concrete, which was taken corresponding to $\varepsilon_c = 0.001$. This approach is adopted by the Canadian Highway Bridge Design Code, Standard CAN/CSA-S6-00 (CSA 2000), which requires a $J$-factor exceeding 4 for rectangular sections. The values of the deformability factor using the above approach are listed in Table 3. For all tested slabs, the $J$-factor is well above the CAN/CSA-S6-00 Code limit of 4 (for rectangular sections). The higher the $J$-factor values, the more ample warning the FRP-reinforced concrete member gives before failure. In other words, the $J$-factor indicates the number of cracks and amount of deflection the FRP-reinforced concrete member will exhibit through load history from service to failure conditions.

Conclusions and recommendations

Public Works and Government Services Canada has decided to investigate the use of FRP composite bars and grids in reinforcing a demonstration area of the structural concrete slabs of a parking garage in Hull (Quebec). This paper presents the proposed design and the laboratory test results of slab prototypes using different FRP reinforcement types and configurations. Based on the experimental test results, the following conclusions can be drawn:

(a) The load carrying capacity of concrete slabs reinforced with composite FRP bars (carbon and glass) was higher than the control slab reinforced with steel (22%–65%) satisfying the same strength and serviceability requirements. In addition, the slabs reinforced with FRP bars failed by concrete crushing, while the control slab and the slab reinforced with carbon grids failed by steel yielding and rupture, respectively, followed by concrete crushing.

(b) At service load level, the calculated cracking control parameter, $\varepsilon_c$, based on actual measured strains and the maximum measured crack width were below the allowable limits of 38 000 N/mm and 0.5 mm, respectively (CSA 2002; ACI 2003).

(c) At service load level, the maximum measured deflections for all tested slabs were below the allowable limits of span/360 (CSA 2002).

(d) The values of the deformability factor, $J$, for the five concrete slabs reinforced with FRP reinforcement were in the range of 14 to 19, which are more than three times the minimum (4 for rectangular sections) required by the Canadian Highway Bridge Design Code (CSA 2000).

(e) The ratios of measured deflections, number of cracks, crack penetration depth, and crack spacing at failure to those at service load level were in the range of 17 to 21, 2, 1.5 to 2, and 0.3 to 0.4, respectively. This, along with the values of the deformability factor, indicates the ample warning that the FRP-reinforced slabs would give before failure.

Test results showed that all FRP reinforcement configurations satisfied both serviceability and strength requirements of the available codes and design guidelines (CAN/CSA-S806-02 (CSA 2002); CAN/CSA-S6-00 (CSA 2000); ISIS-M03-01 (ISIS Canada 2001); ACI 440.1R-03 (ACI 2003)) for concrete structures reinforced with FRP reinforcement. The proposed

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design configurations for the reconstruction of the structural slabs of the Laurier–Taché parking garage can be considered adequate. Based on this work, PWGSC is planning to incorporate the use of FRP into sections of the structural slabs and to monitor the performance over several years.

It is worth mentioning that the slab reinforced with the premanufactured carbon FRP grids was under-reinforced, which is not in agreement with Clause 8.2.1 of CAN/CSA S806-02 (CSA 2002). However, based on the experimental results, the reinforcement configuration of slab S-CCG can be accepted (CAN/CSA S806-02, Clause 5.1.2).

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References


List of symbols

\( A \) effective tension area of concrete surrounding the main tension reinforcement with its centroid coincides with the reinforcement, divided by the number of bars \((\text{mm}^2)\)

\( d_h \) nominal diameter of the bar \((\text{mm})\)

\( E_c \) modulus of elasticity of concrete \((\text{GPa})\)

\( E_t \) modulus of elasticity of FRP bar \((\text{GPa})\)

\( E_s \) modulus of elasticity of steel \((\text{GPa})\)

\( f'_c \) compressive strength of concrete \((\text{MPa})\)

\( f_t \) stress in FRP reinforcement at specified loads, calculated by elastic cracked section theory \((\text{MPa})\)

\( f_{tu} \) ultimate tensile strength of FRP reinforcing bars \((\text{MPa})\)

\( J \) deformability factor

\( I_e \) effective moment of inertia of the beam section \((\text{mm}^4)\)

\( k_b \) bond-dependent coefficient

\( l \) span of the slab \((\text{m})\)

\( M_e \) moment at a service condition such that \(\epsilon_c = 0.001\) \((\text{kN} \cdot \text{m})\)

\( M_{cr} \) cracking moment \((\text{kN} \cdot \text{m})\)

\( M_{sat} \) moment under service load conditions \((\text{kN} \cdot \text{m})\)

\( M_{sus} \) moment under sustained load conditions \((\text{kN} \cdot \text{m})\)

\( M_{ult} \) moment at ultimate condition \((\text{kN} \cdot \text{m})\)

\( M^{\text{ve}} \) maximum moment in the slab between supports \((\text{kN} \cdot \text{m})\)

\( M^{\text{ve}} \) maximum moment over supports \((\text{kN} \cdot \text{m})\)

\( w \) uniformly distributed load on the slab \((\text{kN} / \text{m}^2)\)

\( z \) cracking control parameter

\( \beta_1 \) a factor taken as 0.85 for concrete strength \( f'_c \leq 28 \text{ MPa} \)

For strength \(>28 \text{ MPa}\), this factor is reduced continuously at a rate of 0.05 per each 7 MPa of strength in excess of 28 MPa, but is not taken less than 0.65

\( \beta_2 \) coefficient depending on the degree of fixicity at supports

\( \delta_{\text{max}} \) maximum midspan deflection in an elastic member \((\text{mm})\)

\( \delta_{\text{exp}} \) maximum deflection of the experimentally tested slabs under four-point bending set-up \((\text{mm})\)

\( \delta_{\text{field}} \) maximum deflection of actual continuous parking garage slabs under uniformly distributed dead loads and concentrated wheel loads \((\text{mm})\)
Appendix A. Design example: Design of the structural slabs of the Laurier–Taché parking garage using carbon FRP bars, based on CAN/CSA-S413-94 (CSA 1997) and CAN/CSA-S806-02 (CSA 2002)

Geometry and loading data

The structural system consists of one-way slabs continuous over prestressed concrete girders:

Slab thickness 150 mm
Average spacing between girders (span of slabs) 4.00 m
Width of concrete girders 0.60 m
Clear concrete cover top and bottom) 30 mm
Own weight of wearing surface 0.50 kN/m²
Uniformly distributed service live loads 2.40 kN/m² (NBC, Table 4.1.6.3), or 11.0 kN (NBC, Table 4.1.6.10)

Concrete and carbon fibre reinforced polymer material properties

Specified compressive strength of concrete, \( f'_c \) 35 MPa
Ultimate tensile strength, \( f_{u,ave} \) 1535 ± 45 MPa
Guaranteed tensile strength, \( f_{u} \) (guaranteed) 1535 – 3 × 45 = 1400 MPa
Ultimate tensile strain, \( \epsilon_{fu} \) 0.012
Modulus of elasticity, \( E_f \) 115 GPa
Bar diameter, \( d_b \) 9.5 mm
Bar area, \( A_f \) 71 mm²

Design moments in the deck slab

Service dead load, \( w_{ds} \) 4.03 kN/m²
Service midspan moment (\( M_{ds}^{av} \)) 0.077\( w_{ds} \)\( I \) = 0.077 × 4.03 × (4.00)² = 4.96 kN-m
Service over support moment (\( M_{ds}^{ve} \)) 0.121\( w_{ds} \)\( I \) = 0.121 × 4.03 × (4.00)² = 7.80 kN-m
Factored dead load, \( w_d \)
Factored midspan moment, \( M_d^{ve} \) 1.25 × 4.03 = 5.04 kN/m² (NRC, Clause 4.1.3.2), 6.20 kN-m and factored over support moment, \( M_d^{ve} = 9.75 \) kN-m
Service wheel load, \( P_{ws} \) 11.0 kN (NBC, Table 4.1.6.10)
Service midspan moment (\( M_{ws}^{ve} \)) 0.210\( P_{ws} \) = 9.24 kN-m
Service over support moment (\( M_{ws}^{ve} \)) 0.1811\( P_{ws} \) = 7.96 kN-m
Factored wheel load, \( P_u \)
Factored midspan moment, \( M_u^{ve} \) 1.5 × 11.0 = 16.50 kN (NRC, Clause 4.1.3.2), 13.86 kN-m and factored over support moment, \( M_u^{ve} = 11.95 \) kN-m
The total design midspan service moment, \( M_{s}^{av} \) 4.96 + 9.24 = 14.20 kN-m (Bottom moment)
The total design over support service moment, \( M_{s}^{ve} \) 7.80 + 7.96 = 15.76 kN-m (Top moment)
The total design midspan factored moment, \( M_{u}^{ave} \) 6.20 + 13.86 = 20.06 kN-m (Bottom moment)
The total design over support factored moment, \( M_{u}^{ve} \) 9.75 + 11.95 = 21.70 kN-m (Top moment)

According to ACI 440.1R-03 (ACI 2003), the sustained load is considered as the dead load plus 20% of the live loads. Thus, the factored sustained moment, \( M_{sus} \), is given by

\[
M_{sus} = \text{factored } M_d^{ve} + 0.2 \times \text{factored } M_u^{ve} = 6.20 + 0.2 \times 13.86 = 8.97 \text{ kN-m}
\]

Also, the cracking moment is given by

\[
M_{cr} = \frac{2f_i I_k}{h} = \frac{2 \times 3.66 \times 2.813 \times 10^{-4}}{0.15}
\]
where

\[ I_g = \frac{bh^3}{12} = \frac{1 \times (0.15)^3}{12} = 2.813 \times 10^{-4} \text{ m}^4 \]

and

\[ f_t = 0.6 \sqrt{f_{cu}} = 0.6 \sqrt{35} = 3.55 \text{ MPa (CSA 2002, Clause 8.5.4)} \]

\[ M_{ct} = \frac{2f_t I_g}{h} = \frac{2 \times 3.66 \times 2.813 \times 10^{-4}}{0.15} = 13.75 \text{ kN} \cdot \text{m} \]

**Ultimate limit state: Based on CSA 2002 (Section 8.4)**

Try No. 3 at 125 mm (568 mm²/m)

Effective depth:

\[ d = 115 \text{ mm} \]

\[ \alpha_1 = 0.85 - 0.0015f'_c = 0.8 \]

\[ \beta_1 = 0.97 - 0.0025f'_c = 0.88 \]

\[ \phi A_f f_t = \alpha_{1,0} f'_c ab \]

\[ 0.75 \times 568 \times (115 000 \times \varepsilon_f) = 0.8 \times 0.6 \times 35 \times (0.88 \times c) \times 1000 \]

\[ c = 2916.05 \varepsilon_f \]

and

\[ c / 0.0035 = 115 / (0.0035 + \varepsilon_f) \cdot c \times (0.0035 + \varepsilon_f) = (0.0035 \times 115) \]

\[ \varepsilon_f = 0.010128, f_t = 1164.74 \text{ MPa} \]

\[ \text{the section is over-reinforced} \]

\[ c = 29.53 \text{ mm} \text{ and } a = 0.88 \times 29.53 = 25.99 \text{ mm} \]

\[ c / d = \frac{7}{7 + 2000 \varepsilon_{tu}} \]

\[ c / d = 29.53 / 115 = 0.257 > 7 / (7 + 2000 \times 0.012) = 0.226 \quad \text{OK} \]

\[ M_u = \alpha_{5,b} f'_c ab(d - a/2) = 0.8 \times 0.6 \times 35 \times 1000 \times 25.99(115 - 25.99 / 2) \]

\[ = 44.54 \text{ kN} \cdot \text{m} \]

\[ > 21.70 \text{ kN} \cdot \text{m} \]

\[ > 1.5 M_{ct} = 1.5 \times 13.75 = 20.62 \text{ kN} \cdot \text{m} \quad \text{OK} \]

**Serviceability limit state: Based on CSA 2002 (Section 8.3)**

**Deflection**

The total design service moment \( M_{lte} = 14.20 \text{ kN} \cdot \text{m} \)

(Bottom moment)

**Cracked section properties**

\[ n_f = \frac{E_f}{E_c} = \frac{E_f}{4500 \sqrt{f'_c}} = \frac{115 \times 10^3}{4500 \sqrt{35}} = 4.31 \]

and

\[ \rho_f = \frac{71}{115 \times 125} = 0.494\% \]

\[ k = \sqrt{2 \rho_f n_f + (p_t n_t)}^2 - p_t n_f \]

\[ = \sqrt{2(0.00494)(4.31) + (0.00494(4.31))}^2 \]

\[ - 0.00494(4.31) = 0.19 \]

\[ I_e = \frac{bd}{3}k^3 + n_f A_d d^2(1 - k)^2 = 2.5644 \times 10^{-5} \text{ m}^4 \]

Reduction coefficient for deflection \( \beta_d \) using recommended value \( \alpha_b = 0.5 \)

\[ \beta_d = \alpha_b \left[ \frac{E_f}{E_c} + 1 \right] = 0.5 \left[ \frac{115}{200} + 1 \right] = 0.7875 \]

\[ I_e = \left( \frac{M_{cr}}{M_a} \right) \beta I_g + \left[ 1 - \left( \frac{M_{ct}}{M_a} \right) \right] I_{cr} \]

\[ I_{cr} = \left( \frac{13.75}{14.20} \right)^3(0.7875)(0.00028125) \]

\[ + \left[ 1 - \left( \frac{13.75}{14.20} \right)^3 \right] (25644 \times 10^{-5}) = 19.28 \times 10^{-5} \text{ m}^4 \]

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\[
\delta = \frac{M_{ser}l^2}{12.16 E_c(l_c)} = \frac{(14.20)(4.0)^2}{12.16 \times (27 \times 10^6)(0.001928)} \Rightarrow \delta = 3.23 \text{ mm} 
\]

Limitation \(l/360 = 3600/360 = 10 \text{ mm} \quad \text{OK}\)

**Cracking**

The maximum design service moment, \(M_s = 15.76 \text{ kN} \cdot \text{m} \) (Top moment)

\[
f_t = \frac{M_{ser}}{A_id(1 - k/3)} = \frac{15.76}{5.9166 \times 10^{-4} \times 0.115 \times \left(1 - \frac{0.19}{3}\right)} = 247.36 \text{ MPa}
\]

\(d_c = (h - d) = 35 \text{ mm}\)

\[A = \frac{2d_b}{\text{ Number of bars}} = \frac{2(35)(1000)}{8} = 8750 \text{ mm}^2\]

\[\beta = \frac{h - kd}{d - kd} = \frac{0.150 - 0.19(0.115)}{0.115 - 0.19(0.115)} = 1.375\]

Bond coefficient \(k_b = 1.2\)

Crack control parameter \(z: (8.3.1.1. \text{ CSA/S806-02})\)

\[z = \frac{f_{fr}}{E_s} \sqrt{d_cA} \leq 38 \text{ 000} \]

\[z = \frac{200}{115}(1.2)(247.36)^{1/2}(35)(8750) = 34.796 \text{ N/mm}\]

Crack control parameter, \(z \leq 38 \text{ 000} \text{ N/mm} \) (exterior exposure) \quad \text{OK}

**Design for 2 h fire rating**

The available clear concrete cover to the side far away from fire, \(d_c\), is given by

\[d_c = 150 - 30 - 2 \times 9.5 = 101 \text{ mm}\]

**Design moments and capacity**

In case of fire on the top of the slab, it is assumed that the top reinforcement mat will be totally lost and no moment can be transferred across the supporting beams. Consequently the slab panel will be simply supported on the beams over a span of 4.0 m.

The factored moment

\[= w_d(l/2)(l/4) + P_w(l/2) = 0.125w_d l^2 + 0.251P_w \]

\[= 0.125 \times 5.04 \times (4.0) + 0.25 \times 4.0 \times 16.5 \]

\[= 26.58 \text{ kN} \cdot \text{m}\]

In case of fire on the bottom side of the slab, it is assumed that the bottom reinforcement mat will be totally lost and a major crack will be developed at the middle of the slab panel. Consequently the slab panel will be divided into two parts each is an overhang (2.0 m) from the adjacent panel. However, in the worst case, which is unlikely to occur owing to the continuity of the top reinforcement, a concentrated (wheel load) can be carried by only one overhang (cantilever):

The factored moment

\[= w_d(l/2)(l/4) + P_w(l/2) = 0.125w_d l^2 + 0.251P_w \]

\[= 0.125 \times 5.04 \times (4.0) + 0.25 \times 4.0 \times 16.5 = 43.08 \text{ kN} \cdot \text{m}\]

The ultimate capacity as calculated above

\[= 44.54 \text{ kN} \cdot \text{m} > 43.08 \text{ kN} \cdot \text{m} > 26.58 \text{ kN} \cdot \text{m} \quad \text{OK}\]

**References**


**List of symbols**

- \(b\): width of the slab strip under consideration (mm)
- \(d\): effective depth of tensile reinforcement (mm)
- \(E_c\): modulus of elasticity of concrete (GPa)
- \(E_s\): modulus of elasticity of steel (GPa)
- \(f_t\): stress in FRP reinforcement at service load level, calculated by elastic cracked section theory (MPa)
- \(f_{fr}\): modulus of rupture of concrete (MPa)
- \(h\): height of the slab cross section (mm)
- \(I_{cr}\): moment of inertia of the cracked section of the slab strip (mm\(^4\))
- \(I_e\): effective moment of inertia of the slab strip section (mm\(^4\))
- \(I_g\): moment of inertia of the gross section of the slab strip under consideration (mm\(^4\))
- \(l\): span of the slab (m)
- \(M_a\): applied moment (kN-m)
- \(\alpha_1\): ratio of average stress in rectangular compression block to the specified concrete strength
- \(\beta_1\): ratio of depth of rectangular compression block to depth of the neutral axis
- \(\psi_t\): strain in FRP tensile reinforcement
- \(\rho\): actual reinforcement ratio calculated assuming the effective depth of the slab to be the distance between the top of the slab and the centroid of the lower reinforcement in the main (considered) direction