THREE YEAR PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT WITH GLASS FIBRE REINFORCED POLYMER BARS

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ABSTRACT
The first continuous reinforced concrete pavement (CRCP) using steel bars as reinforcement in Canada was constructed in the province of Québec in 2000. At that time, Project Engineers and experts tried to adapt the design of this type of concrete pavement used in other countries to the severe wet-freeze climate of Québec. In 2005, core samples taken from the first CRCP project revealed that corrosion was active. This caused the engineers to worry about the long-term performance of CRCP. Many alternatives were considered for subsequent projects, and galvanized steel was the alternative that was selected. Another non-corrosive reinforcement alternative is glass fibre reinforced polymer (GFRP) bars, which have been used by the MTQ for many years in concrete bridges. However, this alternative had never been studied in CRCP on a trafficked highway. The first research project of its type involving the application of GFRP bars to actual CRCP slabs was launched September 2006 when construction was completed on 18 test sections on eastbound Highway 40 in Montreal. The performance of these sections has been monitored over the last 3 years, and the many parameters have been evaluated. The feasibility of partial-depth repairs for CRCP using GFRP bars was also demonstrated.

KEY WORDS
CONCRETE PAVEMENT / CONTINUOUSLY REINFORCED CONCRETE PAVEMENT / GLASS FIBRE REINFORCED POLYMER / PERFORMANCE / PARTIAL-DEPTH REPAIR

1. INTRODUCTION

A significant number of transportation infrastructures are in need of repair or replacement, mainly due to the following factors: the use of concrete that is not always suitable to the climate; poor design and construction practices; increase in traffic volume and load; decrease in funds allocated for maintenance; and deterioration resulting from corrosion of pavement reinforcement. However, deterioration due to corrosion of pavement reinforcement has been identified as the most significant factor (Zollinger and Barenberg 1990).

New materials and new construction methods are required in order to protect infrastructures and to avoid this kind of deterioration. One obvious method for controlling this deterioration is to use a material that is naturally resistant to corrosive environments, such as fibre reinforced polymers (FRP). Due to its lower cost, glass FRP (GFRP) has drawn more attention in civil engineering applications as compared to other types of FRP.

GFRP reinforcing bars have recently been used in a number of concrete structures where steel corrosion causes major problems, such as bridges, marine structures, and parking garages. One of the potential applications of GFRP bars is as reinforcement for concrete pavements. Traditional CRCP reinforced with steel bars has been in use for several decades, and many researchers have reported on its performance (Van Breemen 1950; Selezenva and Rao 2004). Until 2006, GFRP had not been used as reinforcement for CRCP, and very little research has been conducted in this area.
The corrosion of steel reinforcement in CRCP, which has been attributed to the development of transverse cracks above and along the length of the reinforcement, is a major concern, especially when subjected to the application of de-icing salt. GFRP bars have a number of advantages, including eliminating steel corrosion, resistance to magnetic fields, and the compatibility of stiffness between GFRP and concrete. However, there are also many differences between steel and GFRP in terms of bond characteristics, strength, elastic modulus, and mode of failure. Therefore, the behaviour of GFRP reinforced elements cannot be extrapolated from that of the steel reinforced elements. Consequently, it is necessary to study the structural performance and to develop the design mechanism for CRCP reinforced with GFRP bars through actual experimental study.

In light of this, along with the continuous collaboration between the Direction du Laboratoire des Chaussées of the Ministère des Transports du Québec (MTQ Pavement Division) and the NSERC Industrial Research Chair in Innovative Fibre Reinforced Polymer (FRP) Composite Materials for Infrastructures at the University of Sherbrooke, the decision was made to implement GFRP technology in one of the CRCP highways in Québec. A 150-m long section of Highway 40 East in Montréal was selected for this pioneer project.

This paper summarizes the research objectives of the project, along with some of the construction details for the 18 full-scale CRCP sections reinforced using GFRP bars. It describes the performance of the GFRP-CRPC test section after 39 months of service, including the progression of cracks, crack width measurements, stresses in the GFRP bars and the concrete, and the temperature inside the CRCP slabs. Repairs were required for some deteriorated zones in the CRCP sections using GFRP. The details of the repair construction and the performance after the following winter months are also presented.

2. EXPERIMENTAL STUDY

This research project focuses on using GFRP bars instead of steel bars in CRCP. The project included design and construction of 18 full-scale CRCP slabs reinforced with GFRP and steel bars over an area of 150 m X 11.3 m covering 3 lanes of A-40 East in Montréal. Monitoring of in-service CRCP-GFRP slabs is required in order to determine their condition and to prevent unsafe conditions. This research project includes the following phases:

- Phase 1: Design, instrumentation, and construction of CRCP-GFRP test sections;
- Phase 2: Monitoring and periodic testing of the CRCP-GFRP sections over 6 years; and
- Phase 3: Analytical modeling in order to predict the broader performance of CRCP-GFRP.

2.1 Research objectives

The main objectives of this study are listed below:

- To investigate the behaviour of CRCP with GFRP bars, and to compare it to reinforced the behaviour of steel bars (in terms of cracking, strains, and stresses);
- To introduce design equations for the use of GFRP reinforcing bars as reinforcement for CRCP;
- To develop an analytical model for CRCP reinforced with GFRP bars using non-linear finite element analysis.

The specific objectives are to determine the effects of:

- the reinforcement ratio, type of reinforcement, spacing of the reinforcements, bar diameter, and pavement thickness on the width and spacing of cracks;
- the depth from the surface of the reinforcements on cracking and stress in the pavement;
- uniformly induced crack spacing on the performance of the CRCP;
- GFRP bond characteristics on cracking and stress in the pavement; and
- the presence of more than one layer of longitudinal reinforcement on the performance of CRCP reinforced with GFRP bars.
The final objective of the project is to develop or adapt a design method for CRCP reinforced with GFRP.

2.2 Design of pavement test sections and instrumentation

A-40 is located in Montréal within the MTQ’s strategic network. The equivalent single axle load (ESAL) estimated for the design period of 30 years was 230 million (concrete). It was determined that a 280 mm (11 in) thick CRCP was required for most of the sections. The slab was constructed on top of a stabilized 100 mm thick cement open graded drainage layer (OGDL).

The next step was to determine the reinforcement required in terms of GFRP bars. Four equations were used to calculate the proposed longitudinal reinforcement ratio: Vetter’s 1933, AASHTO 1972, USDT 1996, and ACI 440.1R-06. Based on the application of these approaches and equations, 1.2% was chosen as the proposed reinforcement ratio. In addition, 6 reinforcement ratios ranging from 0.77% to 1.57% were applied in order to assure the proper longitudinal reinforcement ratio for CRCP slabs reinforced with GFRP bars, as synthesised in Figure 1. The galvanized steel reinforcement ratio used was 0.77%. The detailed reinforcement configuration and investigated parameters are shown in annex A at the end of the paper.

![Figure 1 - Layout and basic design parameters of the test sections](image)

Varieties of sensors were installed in this project in order to monitor the early-age behaviour and repeated load effect on the performance of the CRCP slabs. The set of instruments that were used to collect the data on this project included embedded electrical strain gages for measuring the strain in the concrete and reinforcement (steel and GFRP bars). In addition, thermocouples were used to measure the temperature inside the concrete, and Fibre Optic Sensors (FOS) were used to investigate the difference between electrical gauges and FOS gauges, if any. Four types of FOS were installed in this project, including FOS for measuring strain in the reinforcement (Fabry-perot and Fiber Bragg Grating Sensors), OTP-P Type FOS Thermocouples, and Brillouin sensors. All of the sensors were wired to the Data Acquisition System located at the side of the roadway.

2.3 Material properties and construction

No. 6, 7, and 8 GFRP V-ROD™ vinylester bars were used for the longitudinal and transverse reinforcement of 15 of the sections. These bars are fabricated using the pultrusion process, which generally consists of pulling a continuous roving through a resin bath or impregnator and then into preforming fixtures, where the section is partially shaped and where the excess resin and air is removed. It then enters a heated die, where the section is continuously cured. In order to ensure a proper bond with concrete, the surface of the bars is coated with sand (ISIS Canada, M03-2001, Pultrall 2007). Table 1 shows the mechanical properties of the GFRP bars that were used.
Table 1 - Mechanical properties of GFRP V-ROD™ bars

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Diameter (mm)</th>
<th>Area (mm²)</th>
<th>Elasticity Modulus (GPa)</th>
<th>Guaranteed Tensile Strength* * ( f_{tu}^* ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 6</td>
<td>19.1</td>
<td>285.1</td>
<td>47.6</td>
<td>656</td>
</tr>
<tr>
<td>No. 7</td>
<td>22.2</td>
<td>387.9</td>
<td>46.4</td>
<td>625</td>
</tr>
<tr>
<td>No. 8</td>
<td>25.0</td>
<td>506.7</td>
<td>51.0</td>
<td>611</td>
</tr>
</tbody>
</table>

* \( f_{tu}^* \) = Average tensile strength minus 3 X the standard deviation

Based on MTQ specifications, type IIIA concrete with a maximum water to cement ratio of 0.45 and a slump range of between 20 and 40 mm was used for the demonstration project. The basic mix design for this type consists of 20 mm of coarse aggregates and 340 kg/m³ of Lafarge Tercem-3000 blended silica fume/slag cement. The target compressive strength for this mix is 35 MPa at 28 days.

The construction work for the demonstration project began in August of 2006. The test sections were located at the end of a 6-km reconstruction project on A-40. Anchorages were constructed at both ends of the 150-m sections using at least 3 reinforced lugs. The reinforcement was placed on baskets attached to the OGDL as planned. Between each test section, a 3-m transition area was used to join the different bars or group of bars together. Each shoulder was made of 5-m jointed plain concrete pavement (JPCP). The concrete was mixed near the construction site, delivered to the site on dump trucks, and positioned using a slip-form paving machine. The concrete production rate varied from 60 to 90 m³ per hour. At the end of the paving train, the concrete surface was finished using an Astroturf drag followed by manual transverse tining at 10° relative to the transverse direction. A white-pigmented water-based curing compound was then applied. The central lane was placed first, and the left and right lanes were placed 4 and 8 days later respectively. The reconstructed stretch of A-40, including the demonstration area, was opened to traffic at the end of October 2006. Figure 2 shows two lanes during the concrete paving.

3. PERFORMANCE OF THE PAVEMENT

In 1992, the MTQ launched a pavement performance monitoring program similar to the American Long Term Performance Program (LTTP). The main goals of this program are to improve pavement performance and lifespan and to optimize the resources allocated for road construction and maintenance.
In addition to the surveys mentioned above in connection with the research project, highly detailed performance monitoring for all test sections was planned via the following surveys:

- Distress mapping of all of the 25-m sections;
- Measurement of crack widths and end joints;
- Longitudinal profile measurements (IRI);
- Transverse profile (rut) measurements;
- Coring and sampling;
- Deflection measurements on the slab (FWD);
- Measurement of salt penetration into the concrete.

In order to enable the researchers to investigate the early-age behaviour of the CRCP sections and to judge their performance later in light of the expected long-term results, monitoring of the CRCP sections started on the first day of construction. At the time of writing of this paper, the age of most of the CRCP sections in this project is 39 months. The following sections present the results of the monitoring of the CRCP sections.

3.1 Pavement temperature

In light of the profound effect of temperature on the performance of CRCP slabs, air and pavement temperatures, oscillating in daily and monthly cycles, were measured through all the phases of the project. No major temperature differences were recorded by the thermocouples located at 80 and 230 mm from the top of pavement. The recorded absolute annual temperature variation was approximately 50°C. In addition, A-40 is subjected to severe environmental conditions, including annual temperature variations ranging from -40 to 35°C, more than 40 annual freeze-thaw cycles, more than 3.0 m (118 inches) of snow and up to 1.0 m (39 inches) of rain per year.

3.2 Progression of cracks

Tracking of crack propagation on the surface of the CRCP slabs was carried out in order to monitor the development of the cracking pattern, and to calculate the cracking rate and average crack spacing.

3.2.1 Distress surveys

To date, 18 distress surveys (mapping) have been conducted since construction in 2006. The surveys were started 7 days after casting of the CRCP sections and repeated regularly, approximately every 60 days. The last survey was conducted on February 1, 2010 (age 39 months).

The section labelled SS shown in Figure 3 is a section of CRCP reinforced with steel that was added to the survey in February 2007. This section is located in the right lane, several hundred meters to the east. As it happens, a problem occurred at the anchorage between the end of the 3 steel-reinforced sections (S1, S2, and S3) and the adjacent 15 km long stretch of conventional CRCP containing galvanized steel, resulting in an almost free movement at that end. The cracking was not developing normally in the S sections, and therefore, section SS was added to the evaluated sections.

Figure 3 shows a schematic drawing of the cracks in each section of the test area and section SS, reinforced with conventional steel, at 39 months. The analysis of the cracking surveys is as follows:

- Series A (Reinforcement ratio, varying spacing): After 39 months, the number of groups of cracks in this series was 5 per section, except for section A3 (6). Although the reinforcement ratios vary from 1.05 to 1.32%, the cracking pattern is similar for all of these sections.
- Series B (Transverse reinforcement ratio): Sections B and A2-A3 have similar longitudinal reinforcement ratios, but section B has less transverse reinforcement (spacing of 770 mm instead of 520 mm). Overall, the three slabs seemed to exhibit similar cracking behaviour.

1 One or multiple cracks within a 1-m distance (figure 4)
Figure 3 - Schematic drawing of cracks in all sections

- **Series C (Reinforcement ratio, fixed spacing at 170 mm):** Sections C1 and C2 both showed 6 groups of cracks: C2 had 5 groups of multiple cracks, while C1 had 3. In section C2, a group of very close transverse cracks turned into a distress similar to a punchout, and had to be repaired in the fall of 2009. It should be noted that section C2 is the section with the lowest reinforcement ratio ($\rho = 1.07\%$).

- **Series D (One bar):** Sections D1 and D2 ($\rho = 1.27\%$) are similar in terms of cracking. The majority of the groups of cracks include multiple cracking. In addition, two distresses similar to a punchout had to be repaired in section D1 in the fall of 2009. Unlike sections D1 and D2, sections D3 and D4 exhibit mainly single cracks. These two sections have longitudinal reinforcement ratios of 0.97 and 0.77\% respectively. The trend is that the lower reinforcement ratio with only one bar has fewer multiple cracking groups.

- **Series E (Crack control every 1.2 m):** It is difficult to spot the cracks in this series as a result of the presence of the saw cuts. However, despite the saw cuts, a couple of cracks are present in section E2. In addition, both sections exhibit numerous spalls in the transverse sawed joints, mainly near the wheel paths.

- **Series F and G:** It should be noted that, after 39 months, the ratio of single cracks/groups of cracks for section F (one layer of reinforcement) and section G (two layers of reinforcement), was 50% and 14% respectively. Section G is the only GFRP section that showed angled cracks. This indicates that the slab with two-layers of reinforcement performs better for greater thicknesses of concrete.

- **Series S and section SS:** All of these sections have similar steel reinforcement ratios (0.77\%). In the three sections in the test zone (S1, S2, and S3), the number of cracks in the fall of 2009 was 1, 1, and 3 respectively. However, section SS had 23 cracks. This clearly demonstrates the effect of the poor connection between series S and the adjacent anchorage. In fact, the end of the three S sections was almost free, which practically eliminated all of the restraining forces in these sections. After the full-depth repair of the joint in the fall of 2009, the number of cracks three months later increased to 4, 5, and 4 cracks respectively. Cracking should increase over the coming months.
3.2.2 Cracking rate

The cracking rate can be defined as the length of cracks within each section divided by the area of the section. The cracking rates after 23 and 39 months in service for the 19 test sections are shown in Figure 5.

The E sections are not presented, because it is difficult to see the cracks due to the presence of transverse saw cuts every 1.2 m. An increase in the cracking rate has been observed for all sections between September 2008 and February 2010, except for section D1, where the two partial-depth repairs were carried out in the fall of 2009. For the CRCP test sections reinforced with GFRP, the rate ranged from 0.20 to 0.52 m/m² in September 2008 and from 0.27 to 0.59 m/m² in February 2010. The highest cracking rates are in sections C1 and F, one of which has the highest reinforcement ratio, while the other has the second lowest ratio.
In February 2010, it was observed that the cracking rate was still very low in the three S sections, which are reinforced with steel (0.17 to 0.35 m/m²). This is quite low compared to section SS, which was constructed on another part of the highway. Other new cracks should appear within the coming months, because the problem with the connection between the test zone and the previous CRCP has been corrected. The highest rate was recorded in section SS (1.22 m/m²) which is still higher than the rates obtained from previous CRCP sections of A-40 reinforced with steel, also at 39 months (0.66 and 0.70 m/m² for 0.76% of steel).

3.2.3 Average group crack spacing

The average crack spacing calculated between single cracks is probably not relevant in the case were there are several cracks very close together. Therefore, the average spacing has been calculated between groups of cracks, except for section SS. Figure 6 presents the average group crack spacing (AGCS) at 39 months.

![Figure 6 – Average group crack spacing in February 2010](image)

In terms of the AGCS, most sections in the research project still present abnormally high values three years after construction. The targeted range of crack spacing is normally between 1.07 and 2.44 m. Only sections D3 and D4 are close to that range. Section B has higher AGCS than A2 and A3 (same longitudinal reinforcement but different transverse reinforcement). The AGCS for the S sections is also very high, but was reduced with the arrival of many new cracks subsequent to the full-depth repair. The crack spacing for section SS is below the limit of 1.07 m, and is similar to what was observed on other CRCP steel test sections in our performance program, where no major damage was observed.

3.2.4 Overall cracking behaviour

After more than three years in service, the cracking behaviour of the CRCP section reinforced with GFRP bars is somewhat disturbing. In general, the cracking pattern is not as expected, mainly because the average group crack spacing (AGCS) is still high after three years, and because many cracks have a tendency to concentrate near each other. In addition, the next step of degradation for these closely spaced multiple cracks is similar to a punchout (typical degradation in CRCP). A
punchout usually appears when there is loss of support under the slab where transverse cracks are numerous and closely spaced. Theoretically, this should not occur in this case, with the presence of an OGD under the slab. 3 punchouts had to be repaired in November 2009, and it is expected that others will appear within the coming months.

A loose connection between the end of the test zone and the previous CRCP stretch of A-40 was one of the previously mentioned reasons explaining this cracking behaviour. The repair had a definite effect 3 months later, because many new cracks appeared, mainly within the 50 meters of the joint with the anchorage. In the case of multiple cracking, this may be associated with a higher reinforcement ratio in the transition zone between the sections and at the splicing location. However, these hypotheses cannot be proven, because we do not have the precise locations.

3.3 Crack width

Crack width is not necessarily uniform along the depth of the slab. For the most part, cracks are generally wider at the upper surface of the CRCP slabs compared to the lower surface. This is consistent with the idea of crack formation occurring at the surface as a result of the greater drying shrinkage and temperature contraction (Kohler and Roesler, 2006). In this study, crack widths were measured for all of the CRCP sections at the same time when crack spacing was measured, using a crack comparator. Crack comparators are extremely easy to use, but the reading can be different from one person to another. In order to eliminate human error, only one person was charged with taking crack width measurements.

The average crack widths presented in Figure 7 come from the September 2009 and February 2010 surveys, when the surface temperature was +8° C and -12° C respectively.

All of the average crack widths are higher in winter than in fall. However, some average values are close to the AASHTO design limit for a crack width of 1 mm in sections A2, A3, and D4. This means that some individual values are greater than 1 mm. For example, in section A2, the width of one crack was 1.5 mm, while two other cracks measured 1.25 mm. In section A3, the width of 3 cracks was 1.25 mm. Obviously, the sections with high average crack width values are the ones that have fewer group cracks and more single cracks.
In order to increase the precision of the comparison between crack width surveys, permanent devices were installed in the concrete in November 2007. Crack widths were then measured using a micrometer. Some of the values taken with the micrometer and the crack comparator at the exact same crack are presented in Table 2. The spacing of the nearest cracks is also indicated. The different values taken with the micrometer are less variable than the ones taken with the crack comparator for a temperature decrease of 20°C.

![Table 2 – Comparison of crack width measurements taken with a micrometer and a crack comparator](image)

<table>
<thead>
<tr>
<th>Section</th>
<th>Micrometer (abs. mm)</th>
<th>Crack Comparator (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A2 (spac. ≈ 5 m)</td>
<td>0,404</td>
<td>0,791</td>
</tr>
<tr>
<td>A3 (spac. ≈ 3 m)</td>
<td>1,194</td>
<td>1,644</td>
</tr>
<tr>
<td>C2 (spac. 2.5 and 5.2 m)</td>
<td>0,835</td>
<td>1,234</td>
</tr>
<tr>
<td>D4 (spac. ≈ 2.5 m)</td>
<td>0,380</td>
<td>0,761</td>
</tr>
</tbody>
</table>

3.4 Stresses in CRCP slabs

Stresses in the reinforcement of CRCP slabs are considered to be one of the most important parameters in terms of explaining the performance of CRCP. The purpose of the reinforcement is to limit the contraction/expansion movements of concrete in the CRCP slabs, which in turn affects the crack width and crack spacing at the surface of the CRCP. A number of reinforcement configurations have been used in this study, which means that it is very important to study the stress in the reinforcement throughout the life of the CRCP slabs.

The number of concrete strain gauges varied between 3 and 6 per section, mounted and zip-tied to steel chairs at 30, 110, and 200 mm from the surface of the pavement. The results revealed that, for most sections, higher strains were recorded in the upper strain gauges (30 mm from the pavement surface), which were affected by the temperature variation near the surface of the CRCP slab. In addition, the minimum strains were found in series E (section E1), where induced cracks were applied, which in turn reduced the strain throughout the slab.

In general, the strains that affect the reinforcement of CRCP are somehow complicated issues, because they depend on many parameters, including as the age of the CRCP slabs and the position of the crack on the surface of the slab relative to the position of the strain gage. For many slabs, the strains had a parallel or contrary relationship with the pavement temperature:

![Table 3 – Tensile strain of GFRP bars and concrete](image)

<table>
<thead>
<tr>
<th>Series</th>
<th>Maximum measured Tensile Strain (µε) of GFRP bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>750 (A3) to 5000 (A4)</td>
</tr>
<tr>
<td>B</td>
<td>1000</td>
</tr>
<tr>
<td>C</td>
<td>750 (C2)</td>
</tr>
<tr>
<td>D</td>
<td>3500 (A4) and 4500 (A1 and A2)</td>
</tr>
<tr>
<td>E</td>
<td>1750 (E1) and 3000 (E2)</td>
</tr>
<tr>
<td>F and G</td>
<td>300 (F) and 3500 (G)</td>
</tr>
</tbody>
</table>

With the exception of section A4 (5000 µε), it was observed that the maximum tensile reinforcement strains were recorded in the four sections of series D (3500 to 4500 µε). These values are approximately 22 to 29% of the strain limit for GFRP bars, which is still less than the allowable limit according to CAN/CSA standard S806 (30%). In addition, these values are decreasing with time as a result of the formation of new cracks. The minimum tensile reinforcement strains were measured in sections F (225 µε) and S1 (275 µε).
3.5 Core sampling

In May 2009, a complete investigation of the test sections was conducted. Core sampling was carried out in order to investigate some cracks with high severity distresses, such as in section D1 (punchout). Other core samples were taken at single cracks (A2, A4 and G sections) and in between cracks (A4 section). The two core samples taken at 63 m in section D1 (i.e.: the middle lane) revealed that the concrete is cracked from top to bottom, and that there is a cracking plane at the GFRP layer for the core sample taken closest to the punchout, as shown in Figure 8.

![Figure 8 – Core samples and punchout in D1 section at 63 m](image)

In both single cracks in sections A2 and A4, the concrete pavement was cracked from top to bottom, and unexpectedly, there was a cracking plane at the GFRP layer, as shown in Figure 9. We did not investigate the area of the horizontal cracking plane. In sections A4 and G (two-layer reinforcement), the core samples did not reveal any horizontal cracking planes.

![Figure 9 – Core sample from a single crack in section A4](image)

4. CRCP REPAIR USING GFRP BARS

Repairing damaged concrete pavement for slabs reinforced with GFRP bars is uncertain when a type of repair that will reach the GFRP layer is contemplated. An internal study that was conducted
in the laboratory in 2007 revealed that trying to uncover GFRP bars carefully with a very small jackhammer still causes significant damage to the bars (Hovington and al., 2007). Some chips in the bars reduced the transverse section of the bar by up to 50%.

Meanwhile, repair of the loose connection between the end of the test section and the previous CRCP stretch of A-40 had already been planned. It was decided to repair the punchouts as well, because some pieces of concrete were starting to come out, and the maintenance unit had to go and patch the deteriorated areas frequently. This was a great opportunity to determine whether repair of concrete reinforced with GFRP bars is possible, and to determine which precautions must be taken in order to avoid causing them to deteriorate.

The work was carried out during the first two weekends of November 2009. The right and centre lanes were repaired during the first weekend, while the left lane was repaired the next weekend. Three partial-depth repairs were carried out in conjunction with the full-depth repair at the east-end connection. For the partial-depth repairs, two saw cuts were made along the width of the lane, approximately 90 mm deep separated by 1 to 1.5 m, in order to include all of the groups of cracks. The jackhammer workers were very careful when they approached the depth of the GFRP bars.

Figure 10 shows the demolition of the concrete and one of the cavities ready to be filled with the patching material.

After the concrete was removed, the GFRP reinforcement bars appeared in three forms: undamaged, chipped, and squeezed. Amazingly, most of them were in very good condition (Figure 10: right-hand picture) thanks to the care taken by the workers.

Most of the damaged bars were at the location of the punchout, as shown in Figure 11. The chipping of the bars typically originated from the removal of the concrete in the repair area using the jackhammers. On the other hand, the squeezing of the bars is related to impact loading from the traffic on the crushed pieces of concrete in the punchout.

As can be seen, some crushed pieces of concrete were left in place in the area of the punchout. The repair areas were meticulously cleaned, and a Delpatch™ product was mixed and poured in these areas. The only problem was the levelling of the patching material used over a large surface area.
Figure 11 – Damaged GFRP bars (chipped and squeezed)

Figure 12 shows one of the repaired areas in July 2010. The repaired areas are performing very well after one winter season.

Figure 12 – One of the repaired areas (July 2010)

5. SUMMARY AND CONCLUSIONS

This paper describes the performance of 18 full-scale CRCP test sections reinforced with GFRP bars located along highway 40 East in Montréal after three years in service. This research project was conducted by the Ministère des Transports du Québec in collaboration with the Université de Sherbrooke for the purpose of investigating the various parameters known to affect the performance of CRCP. The research project focuses on using glass fibre reinforced polymer (GFRP) bars instead of steel bars in CRCP.

The installation of a short experimental test section of CRCP with GFRP reinforcement should allow us to verify the feasibility of construction using this type of reinforcement and the adequate initial behaviour of the pavement (i.e.: no innate problems with the slab). It has been clearly demonstrated that these two objectives were achieved, but up to now, only few preliminary trends have been established. The following observations were made after studying the results of the test sections by series:
- Series A: Increasing the reinforcement ratio from 1.05 to 1.27% did not make a noticeable difference in the cracking data, which is not what we expected. The crack widths in winter and the stresses in section A4 are the highest among the GFRP sections.
- Series B: Section B exhibits the highest crack spacing (AGCS) among the GFRP sections. Decreasing the transverse reinforcement may have an effect on the cracking of the slabs.
- Series C: Despite the 0.5% difference in the reinforcement ratio between sections C1 and C2, there is little variation in crack spacing (AGCS) and crack width. As expected, section C1, which has the highest reinforcement ratio (1.57%), also has the highest cracking rate among the GFRP sections.
- Series D: The use of single bars with a lower reinforcement ratio presents fewer multiple cracks. In addition, the crack rate and crack spacing decrease with the reinforcement ratio. However, the lowest reinforcement ratio exhibits the highest crack widths. These trends are closer to the trends observed for CRCP with steel reinforcement.
- Series E: It is not easy to determine the number of cracks present in the slab as a result of the saw cuts for crack control. Despite the saw cuts, 1 crack appeared in section E2.
- Series F and G: For greater slab thicknesses, two-layers of GFRP bars performed much better in all of the aspects that were evaluated.
- Series S and SS: Comparison with the GFRP sections is not accurate, firstly because the behaviour of the S sections was not as expected during the first three years, and secondly because section SS exhibits an abnormally high cracking rate.

Despite these preliminary trends, the observed behaviour after a few years leaves us perplexed. In fact, two phenomena that emerged during the second year of commissioning were unexpected: the presence of several distresses similar to a punchout, and the increased spacing of cracks or groups of cracks. The various reports have not yet explained what led to these types of cracking degradations. Certain assumptions and findings have been tabulated in an effort to explain these trends:

- The configuration and the very short length of the test sections may not fairly represent the behaviour of a longer stretch.
- The many areas of transition between the sections may have induced undesirable constraints and modified the cracking pattern.
- The presence of 5-m long jointed plain concrete slabs on the shoulders definitely induced stresses near the joints and created sympathy cracks in the test sections (80% in the left lane and 65% in the right lane).
- The loose connection between the end of the test section and the previous CRCP stretch definitely played a major role in the development of the cracking in the adjacent sections of steel reinforced CRCP (few cracks).
- The presence of a 100-mm thick OGDL theoretically increased the total thickness of concrete pavement, which decreased the reinforcement ratio.

A test section was also constructed in 2007 along Route 9 in Martinsburg, West Virginia. The 254-mm thick slab was constructed on a base of 100-mm OGDL and 274-mm treated cement. A 610-m control section of the same length was built using steel. Reinforcement percentages for the section using steel and the one using GFRP were 0.7% and 1.12% respectively. No. 7 GFRP bars were used, spaced every 152 mm. The data available after 6 months reveal greater average crack spacing for the section using GFRP (3.8 m compared to 2.1 m the section using steel). It was also observed that the OGDL increases the thickness of the concrete pavement. Despite the fact that the results published in this section are fragmentary, the trends are similar to ours in terms of crack spacing.

In light of this, more investigations are required in order to study the interaction behaviour between concrete and GFRP reinforcement and its effect on the general behaviour of CRCP slabs. For example:

- The possible delaminating of the concrete at the bar level under traffic effects resulted in punchouts. This horizontal delaminating sometimes appeared at multiple cracks and sometimes at a single crack. What is the extent of this phenomenon?
- Taking into account the excess width of some of the cracks, what is the load transfer efficiency, and how will these cracks deteriorate over time (intrusion of incompressible)?

Based on the construction details and the preliminary results, the following conclusions can be drawn:

- The results of this demonstration project will contribute to implementing the use of GFRP bars in concrete pavement. In addition, this project will introduce pioneer results to the literature pertaining to the use of GFRP bars in CRCP.
- No problems were encountered in the production, transportation, handling, or construction of CRCP reinforced with GFRP.
- The performance of the experimental sections are yet to be explained in relation to the presence of several distresses similar to a punchout, and the increased spacing of cracks or groups of cracks. More time is needed to evaluate the behaviour of the slabs.
- Surface repair of CRCP using FRP bars is possible, but workers must be extremely cautious when breaking the concrete around the GFRP bars.

Over the next months, we hope to generate interesting data and discover more accurate trends demonstrating the relation between the concrete and the GFRP reinforcement. With respect to West Virginia's project, we may need a longer stretch of CRCP with GFRP bars in order to be able to better understand the performance. We also hope to be able to create a design method and conduct a Life Cycle Cost Analysis for a variety of options.

REFERENCES


VETTER, C. P. (1933) “Stresses in Reinforced Concrete Due to Volume Changes,” ASCE, Vol. 98, pp. 1039-1050


ZOLLINGER, D. G., AND BARENBERG, E. J. (1990) “Continuously Reinforced Concrete Pavements: Punch outs and other Distresses and Implications for Design,” Research project IHR-518, Transportation Research Laboratory, University of Illinois at Urbana – Champagne
## Annex A - Reinforcement configuration (final) and investigated parameters for the CRCP research project, Highway 40- East (Montréal)

<table>
<thead>
<tr>
<th>Series</th>
<th>Investigated Parameters</th>
<th>Section</th>
<th>Slab Thickness (mm)</th>
<th>Longitudinal Rein. Ratio (%)</th>
<th>Depth of Rein. (mm)</th>
<th>Longitudinal Reinforcement</th>
<th>Transverse Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Reinforcement Ratio (Different Spacing)</td>
<td>A1</td>
<td>280</td>
<td>1.05</td>
<td>100</td>
<td>2xGFRP-20M @ 195 mm</td>
<td>1xGFRP-20M @ 520 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A2</td>
<td>280</td>
<td>1.16</td>
<td>100</td>
<td>2xGFRP-20M @ 176 mm</td>
<td>1xGFRP-20M @ 520 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A3</td>
<td>280</td>
<td>1.16</td>
<td>100</td>
<td>2xGFRP-20M @ 176 mm</td>
<td>1xGFRP-20M @ 520 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A4</td>
<td>280</td>
<td>1.32</td>
<td>100</td>
<td>2xGFRP-20M @ 154 mm</td>
<td>1xGFRP-20M @ 550 mm</td>
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<tr>
<td>B</td>
<td>Transverse Reinforcement Ratio</td>
<td>B</td>
<td>280</td>
<td>1.16</td>
<td>100</td>
<td>2xGFRP-20M @ 176 mm</td>
<td>1xGFRP-20M @ 770 mm</td>
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<tr>
<td>C</td>
<td>Reinforcement Ratio (Fixed Spacing)</td>
<td>C1</td>
<td>280</td>
<td>1.57</td>
<td>100</td>
<td>2xGFRP-22M @ 176 mm</td>
<td>1xGFRP-20M @ 510 mm</td>
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<td></td>
<td></td>
<td>C2</td>
<td>280</td>
<td>1.07</td>
<td>100</td>
<td>1xGFRP-25M @ 168 mm</td>
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<tr>
<td>D</td>
<td>One Bar</td>
<td>D1</td>
<td>280</td>
<td>1.27</td>
<td>100</td>
<td>1xGFRP-25M @ 143 mm</td>
<td>1xGFRP-20M @ 520 mm</td>
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<tr>
<td></td>
<td></td>
<td>D2</td>
<td>280</td>
<td>1.27</td>
<td>100</td>
<td>1xGFRP-25M @ 143 mm</td>
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<tr>
<td></td>
<td></td>
<td>D3</td>
<td>280</td>
<td>0.97</td>
<td>100</td>
<td>1xGFRP-22M @ 143 mm</td>
<td>1xGFRP-20M @ 520 mm</td>
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<td></td>
<td></td>
<td>D4</td>
<td>280</td>
<td>0.77</td>
<td>110</td>
<td>1xGFRP-20M @ 130 mm</td>
<td>1xGFRP-20M @ 530 mm</td>
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<td>E</td>
<td>Crack Control (Saw Cut every 1.2 m)</td>
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<td>280</td>
<td>1.16</td>
<td>130</td>
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<td></td>
<td>E2</td>
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<tr>
<td>F</td>
<td>Thickness</td>
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<td>350</td>
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<td>100</td>
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<td>1xGFRP -20M @ 400 mm</td>
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<tr>
<td>G</td>
<td>Two Layers</td>
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<td>100 &amp; 200</td>
<td>1xGFRP-20M @ 176 mm For Each Layer</td>
<td>1xGFRP -20M @ 820 mm For Each Layer</td>
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<td>Steel</td>
<td>S1</td>
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<td>0.77</td>
<td>110</td>
<td>1x20M @ 137 mm</td>
<td>1x15M @ 700 mm</td>
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<tr>
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<td></td>
<td>S2</td>
<td>280</td>
<td>0.77</td>
<td>100</td>
<td>1x20M @ 137 mm</td>
<td>1x15M @ 700 mm</td>
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<tr>
<td></td>
<td></td>
<td>S3</td>
<td>280</td>
<td>0.77</td>
<td>130</td>
<td>1x20M @ 137 mm</td>
<td>1x15M @ 730 mm</td>
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