

Field and Laboratory Performance of Cementitious Partial Depth Repair Materials in Cold Climates

Haithem Soliman, EIT¹ and Ahmed Shalaby, P.Eng.²

Abstract

Incompressible debris accumulating in pavement joints or cracks restricts the free expansion of pavement slabs during warm weather and generates excessive compressive stresses along the joint or crack faces. These stresses cause deterioration and spalling of slab corners and edges and accelerate the rate of pavement degradation. Partial depth repair is a preventative maintenance treatment which is commonly used early in the pavement service life to repair spalls and shallow deterioration and restore the intact pavement surface typically before excessive deterioration occurs. The selected repair material must be compatible with the environmental and load conditions. Replacing the deteriorated concrete with a durable material restores the structure integrity, improves the ride quality, and reduces moisture infiltration to subsurface layers of the pavement. The objective of this paper is to evaluate the performance of several rapid setting cementitious concrete repair materials in cold climates and to develop selection criteria for partial depth repair materials. Six repair materials were applied to a test section on a major arterial road in the City of Winnipeg in the summer of 2010. Pre- and post installation detailed condition surveys were conducted at the repair sites and the field evaluation will continue for the next two years. The field performance of the repair materials was evaluated according to: presence of transverse and longitudinal cracks, separation between the material and concrete slab, surface finish of the repair area, and deterioration of the repair material. Laboratory tests were conducted to evaluate the impact of freeze-thaw and wet-dry cycling on the bond strength between repair materials and regular concrete. Concrete cylinders were prepared in the laboratory. The prepared cylinders had a concrete-repair material interface slanted at 30 degrees from the loading axis. The specimens were subjected to freeze-thaw and wet-dry cycling separately. The degradation of the bond strength between repair material and concrete was evaluated from the compressive testing of the bond cylinders at different levels of freeze-thaw and wet-dry conditioning. Results of laboratory and field evaluation will be used to develop performance-based selection criteria for cementitious partial depth repair materials.

Introduction

Partial depth repair (PDR) is a preventative maintenance treatment for portland cement concrete (PCC) which is commonly used early in the pavement service life to repair spalls and shallow deteriorations of pavement slabs. Partial depth repairs are mainly located along the joints of the slab, but can be placed anywhere in the slab to replace deteriorated concrete. The cause and depth of the spall are limitations to using partial depth repair to ensure good performance and cost effectiveness of the repairs. Partial depth repair should not be used when spalling is deeper than the top third of the slab thickness or when pavement has large amounts of deterioration which are signs that the pavement has little service life left (Wilson et al. 1999a).

Partial depth repair consists of removing the deteriorated concrete and replacing it a durable material to restore the structure integrity, improves the ride

¹ Doctoral Student, Department of Civil Engineering, University of Manitoba, Canada,
Email: umsolimh@cc.umanitoba.ca

² Professor and Head, Department of Civil Engineering, University of Manitoba, Canada,
E-mail: shalabya@cc.umanitoba.ca

quality, and reduces moisture infiltration to subsurface layers of the pavement. The repair material when installed with a good quality control can have good performance for more than 5 years of service (Wilson et al. 1999a). There are different procedures for preparation of repair area and removal of deteriorated concrete (Soliman et al. 2011). Using different procedures for preparation of repair area does not have a significant effect on the performance of partial depth repairs if it meets the requirements of the repair material manufacturer (Wilson et al. 1999a).

Failure of partial depth repairs is commonly due to the improper selection of the repair material and the incompatibility between the repair material and original concrete (Wilson et al. 1999b). There are three main types of repair materials: cementitious, polymer, and bituminous repair materials. The agency selects the type of the repair material that fits the necessary performance criteria. The physical properties of the repair material, the bond strength between the material and original concrete, and the durability of the repair material should be evaluated before approving the repair material for use (American Concrete Pavement Association 2008). In cold climates, repair materials are subjected to a large annual temperature differential that may exceed 80°C. For this reason, field studies are conducted to evaluate the field performance of repair materials and approve it for use which does not provide a timely response to progressive market changes and the availability of new products.

Several laboratory testing methods have been proposed to evaluate the compatibility between the repair material and concrete. The coefficient of thermal expansion (CTE) is one of the material properties which can affect the compatibility between the repair material and concrete. The difference between the CTE of the concrete and the repair material may cause an existing transverse crack to open or close with changing temperature conditions (Markey et al. 2005). Pattnaik and Rangaraju (2007) investigated the compatibility between cementitious repair materials and concrete using a modified ASTM C 78 test procedure. Composite simple beams were prepared from the repair materials and regular concrete. Analysis of flexural strength and load-deflection curves obtained from third point loading showed that significant differences in the strength between the repair material and substrate concrete cause incompatible failures. High drying shrinkage of the repair materials was also found to cause incompatible failure.

Al-Ostaz et al. (2010) used the slant shear test to evaluate the bond strength between repair material and concrete. The slant shear test was conducted in accordance with British Standard BS EN 12615: 1999 which is equivalent to the slant shear test in ASTM Standards ASTM C 882-91. Two types of repair materials were evaluated in this study, cementitious and polymer materials. Slant shear tests were conducted on unconditioned specimens and specimens that were subjected to thermal cycling from +12.8°C to +51.7°C which simulate the fluctuations of temperature in extreme summer days. For unconditioned specimens, polymer repair materials had higher bond strength to concrete than cementitious repair materials. After being subjected to thermal cycling, polymer repair materials showed higher reduction of bond strength than cementitious repair materials.

This paper reports on the evaluation of the laboratory and field performance of six cementitious repair materials. The repair materials were installed in two test sections in 2010 (Soliman et al. 2011). Laboratory testing was conducted in order to evaluate the bond strength between the repair materials and concrete. Results of laboratory and field performance will be used to evaluate the compatibility of the repair materials with the construction materials and the environmental conditions in

cold climates. The research will provide performance-based selection criteria for PDR materials. These selection criteria will be a cost-effective accelerated alternative to full-scale field studies, and provide a timely response to progressive market changes and the availability of new products.

Field Installation of Repair Materials

Six cementitious repair materials were installed in two test sections of Portage Avenue in Winnipeg, Manitoba as shown in Figure 1 (Soliman et al. 2011). Both test sections consist of a two way divided road with 3 traffic lanes and one parking lane in each direction. The middle lane of both test sections was selected for installation of the repair materials. The repair materials were installed 6 m away from the adjacent intersection. The posted speed limit is 60 km/h and the average weekday daily traffic is 34,800 vehicles over the six lanes of traffic. The traffic observed along both test sections was 91% passenger cars. The existing pavement consisted of a Jointed Plain Concrete Pavement with a width of 3.70 m, a thickness of 255 mm, and an apparent maximum aggregate size of 25 mm. The pavement was placed on a limestone base.

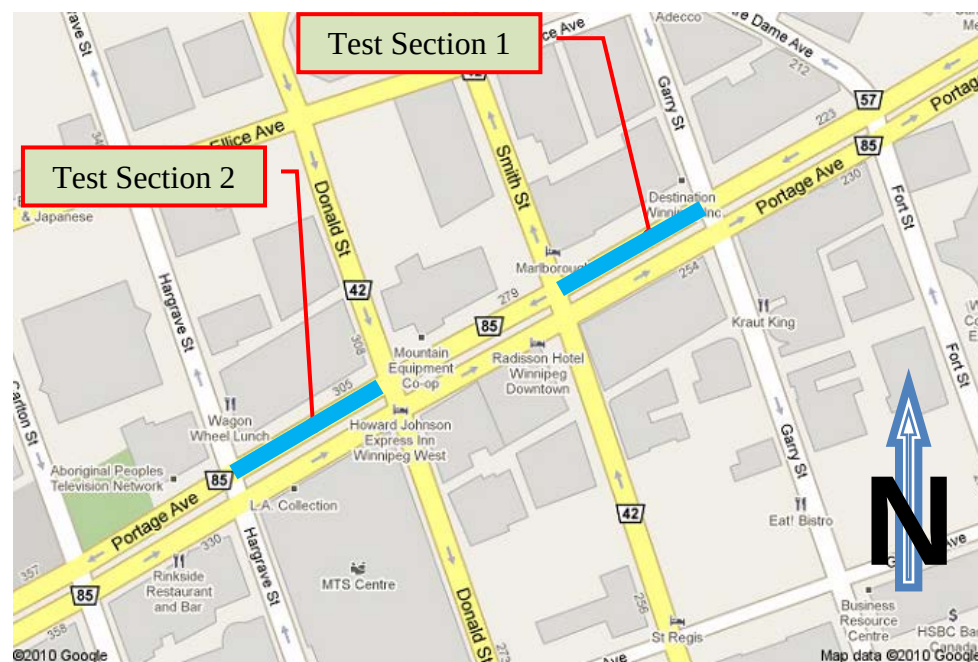


Figure 1: Location of Test Sections

All repairs areas in both test sections were located along the longitudinal or transverse joints. The severity level of the spalls was more significant along transverse joints than longitudinal joints, as shown in Table 1. The severity level of the spalls along longitudinal joints was predominantly low for both test sections. On the other hand, the majority of transverse joints along test section 1 had low severity while test section 2 had predominantly moderate severity. Figure 2 shows some distressed joints before repairing.

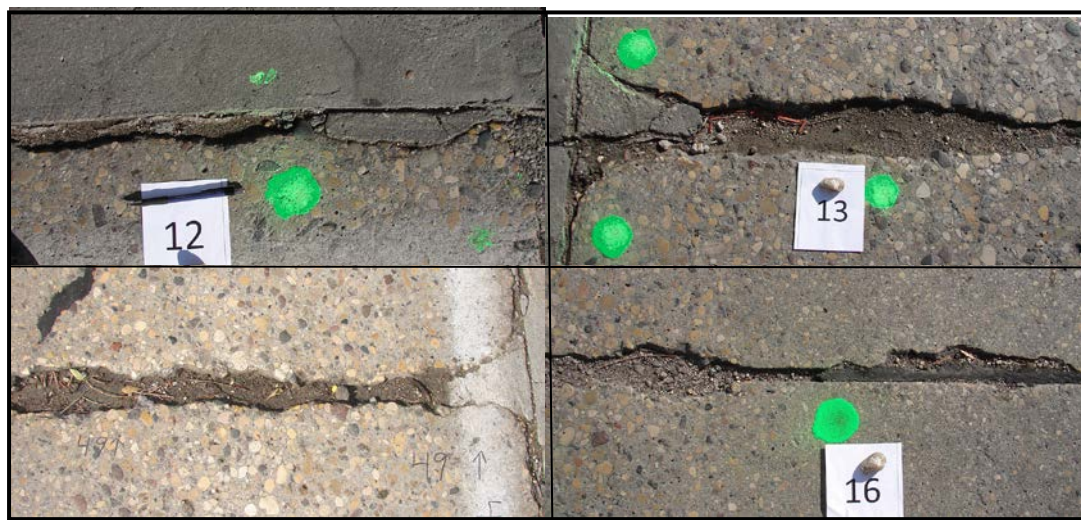
Table 1: Number of Spalls Grouped According to Severity Level

Test Section	Joint Type	Number of Spalls		
		Low ¹	Moderate ²	High ³
1	Transverse	27	12	1
	Longitudinal	26	1	-
2	Transverse	4	18	-
	Longitudinal	25	1	-

¹Low: Spalls less than 75 mm wide, measured to the center of the joint, with loss of material, or spalls with no loss of material and no patching.

²Moderate: Spalls 75 mm to 150 mm wide, measured to the center of the joint, with loss of material.

³High: Spall greater than 150 mm wide, measured to the center of the joint, with loss of material.

**Figure 2: Distressed Joints before Repairing**

The saw and patch method was used to remove the deteriorated concrete (Soliman et al. 2011). This method consisted of saw cutting the marked edges of the repair to a depth of 40 mm from the pavement surface using a concrete cut-off saw mounted on a cutting cart. Once the repair boundaries were cut, the concrete was removed by a 11.3 kg jackhammer. The concrete was removed by first jackhammering near the center of the repair area and working outwards to the saw-cut. Then the repair surface along the bottom edge of the saw-cut was jackhammered in order to fully expose the vertical surface of the cut. The concrete within the repair area was removed to the bottom of the saw cut or 10 mm below visually sound and clean concrete. The cleaned concrete surface was at a typical 10 to 45 degrees angle from the horizontal as shown in Figure 3.

Once the concrete had been removed, the repair area was thoroughly sandblasted to remove cement dust, dirt, and other debris present. Afterwards, the repair area was air blown to remove any particles left by the sandblasting. This preparation provided a clean surface for the repair material to adhere and bond. The final joint preparation was to mist the surface of the repair area with water immediately before the placement of the repair material. This was done to reduce the tendency of the concrete to attract water from the repair patch and reduce the effectiveness of the patch.

Three repair materials were placed per each test section. Table 2 shows the technical data of the repair materials. Aggregate extender, well graded 10 mm rounded gravel, was added to 4 repair materials as shown in Table 2. The repair materials were mixed according to the manufacture's specifications and recommendations. After being poured into the repair area, the repair material was vibrated using a 25 mm head vibrator. The repair area was finished using a trowel to re-establish the vertical profile of the joint. Some repair materials had low workability which resulted in poor surface finish. A curing compound was applied after finishing the surface of the repair area.

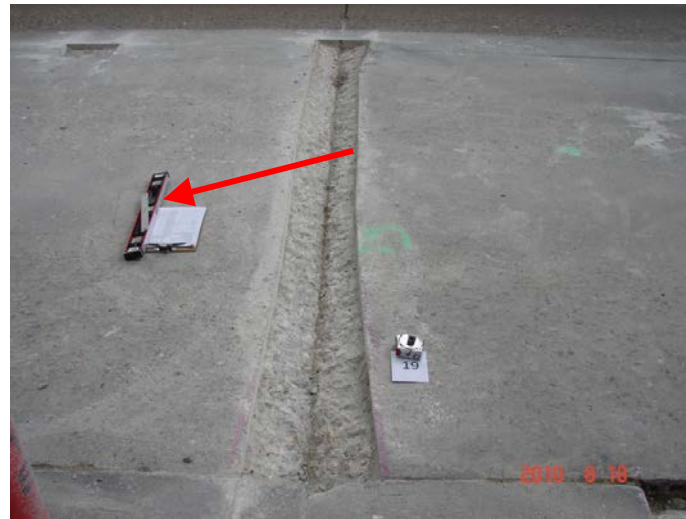


Figure 3: Typical Cleaned Concrete Surface

Field Performance of Repair Materials

The field performance of the repair materials were evaluated one, six, nine, and thirteen months after installation. The following criteria were used to evaluate the performance of the repair materials:

- Transverse (shrinkage) cracking
- Longitudinal cracking
- Cracking at the saw-cut surface
- Surface finish after installation
- Spalls
- Surface wear

Table 2: Summary of Technical Data of the Repair Materials

Repair Material	A	B	C	D	E	F
Composition [% by weight]						
Portland cement	-	✓	30 - 60	5 - 10	10 - 30	7 - 13
Hydraulic cement	✓	-	10 - 30	-	-	-
Alumina cement	-	-	-	10 - 30	-	-
Silica sand, crystalline	✓	✓	40 - 70	30 - 60	-	> 60

Silicon dioxide	✓	-	-	-	-	-
Titanium dioxide	-	-	-	-	-	0.1 - 1
Formaldehyde	-	-	< 0.1	-	-	-
Calcium sulfate	-	-	-	1 - 5	-	-
Tricalcium aluminate	-	-	-	0.5 – 1.5	-	-
Borax	-	-	-	-	-	1 - 5
Various oxides (Al, Ca, and/or Mg)	-	-	-	-	5 - 10	-

Physical and Chemical Properties						
pH	-	-	12	11 - 13	8.5	-
Specific gravity	2.7	2.6 - 3.2	2.7	2.0 - 2.3	2.63	2.75
Water Content [L]	2.13 - 2.84	2.60 - 2.84	2.84	2.84	1.89	1.60 - 1.77
Mixing Time [min]	4 - 5	8	4	4	7	4
Extension *	80%	-	100%	80%	-	50%

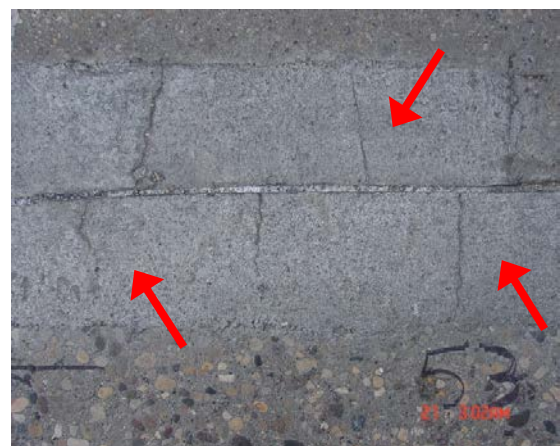
* Coarse aggregate extension by weight of repair material per bag

✓ Chemical composition by weight not provided but stated

Transverse cracking, shown in Figure 4, are the cracks in the transverse direction of the repair area. The severity of the transverse cracking was categorized as low, medium, or high according to the spacing between cracks along the repair area. Longitudinal cracking, as shown in Figure 5, are the cracks in the longitudinal direction of the repair area. Cracking at the saw-cut surface, as shown in Figure 6, are the initiation of separation between the repair material and the original concrete. Surface finish, as shown in Figure 7, represents the regularity of the repair surface after installation of the repair material. Spalls, shown in Figure 8, are the chipping off of pavement in the repair area. Surface wear, as shown in Figure 9, represents the pop-out of aggregate and fines from the repair materials due to traffic and weathering.



a) Low severity cracking



b) High severity cracking

Figure 4: Transverse Cracking in the Repair Area



Figure 5: Longitudinal Cracking in the Repair Area



Figure 6: Cracking at the Saw-Cut Surface in the Repair Area



Figure 7: Repair Areas with Poor Surface Finish

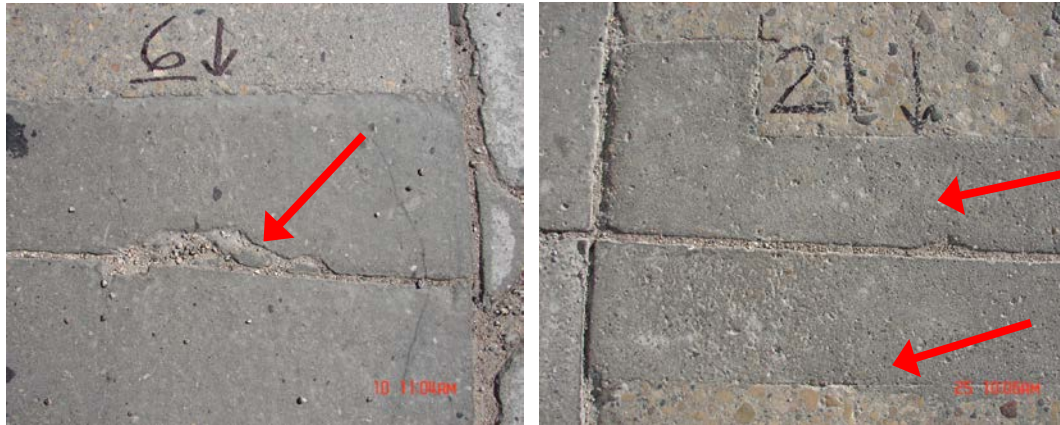


Figure 8: Repair Areas with Spalls Figure 9: Repair Areas with Surface Wear

Figures 10 to 15 show the frequency of the distresses for the repair materials one, six, nine, and thirteen months after installation. All the repair materials had good surface finish except material A, where 32% of these patches had a poor surface finish. Areas repaired with materials E and F had lower percentage of transverse cracking than the remaining materials. Almost all the transverse cracking was observed one month after installation. Repair materials that had high water content during mixing experienced higher percentage of transverse cracking. All the repair material had no, or very small percentage, of longitudinal cracking except for repair material B. The longitudinal cracking near the wheel path can be attributed to the structural instability of the patched area and are not related to the type of repair material.

After one month, materials D and F had the lowest percentage of cracking at saw-cut surface, while material B had the highest percentage of cracking. After thirteen months, material D was the only material that maintained low percentage of cracking at saw-cut surface. For surface wear, 33% of the patches repaired with material D showed signs of surface wear after six months; this percentage ranged from 62% to 89% for the remaining repair materials. After thirteen months, all repaired areas showed signs of surface wear. The percentage of patches with spalls ranged from 2% to 16% after six months. After thirteen months, the percentage of patches with spalls ranged from 8% to 26%.

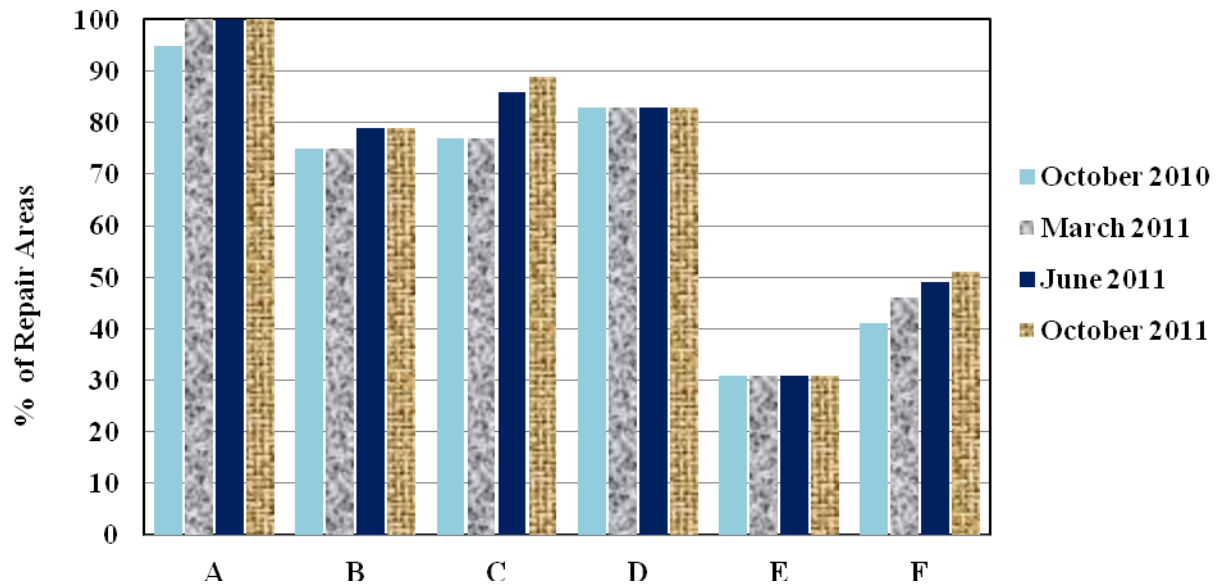


Figure 10: Percentage of Repair Areas with Transverse Cracking

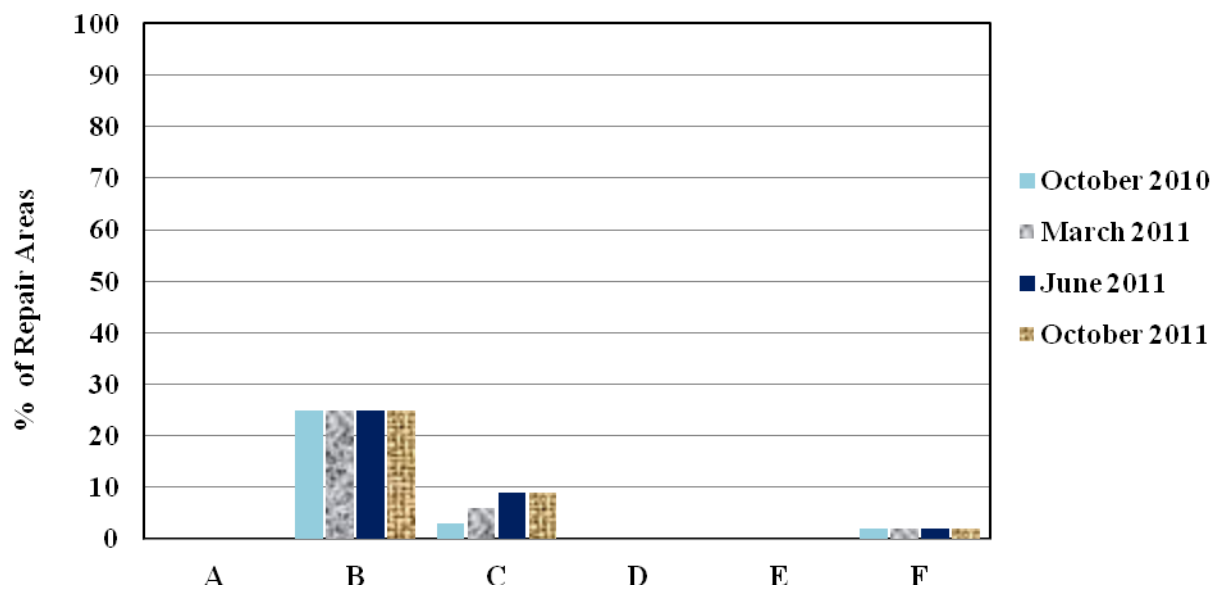


Figure 11: Percentage of Repair Areas with Longitudinal Cracking

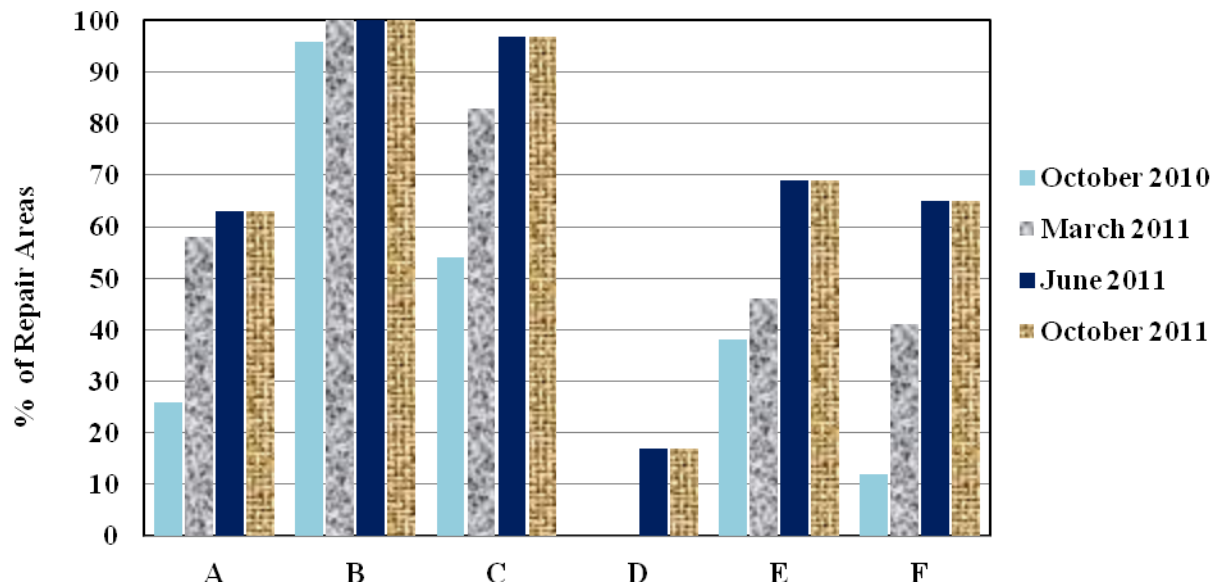


Figure 12: Percentage of Repair Areas with Cracking at Saw-Cut Surface

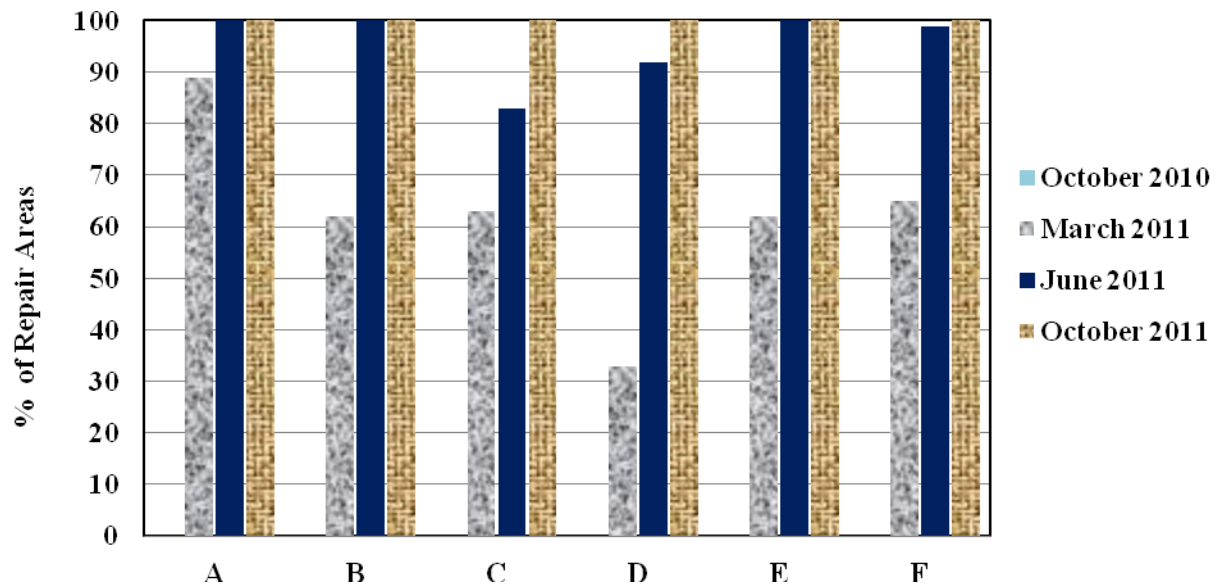


Figure 13: Percentage of Repair Areas with Surface Wear

Laboratory Performance of Repair Materials

The purpose of the laboratory testing is to evaluate the compatibility between the repair material and the concrete substrate along with the resistance of the repair material to environmental conditioning. The laboratory evaluation includes the six repair materials installed in the field. The laboratory evaluation of repair materials includes: compressive strength, freeze-thaw durability of bond strength, and wet-dry durability of bond strength.

The compressive strength of the repair materials was tested according to ASTM C 873: *Standard Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds* (ASTM 2008). Three 3×6 inches cylindrical samples of each repair material were cast and tested until failure. The samples were left curing at room temperature and 100% humidity for 28 days before being tested. Table 3 shows the compressive strength of the six repair materials obtained from the

laboratory compacted samples. Samples that exhibited strength variation greater than 30% were excluded from the calculation of the average compressive strength.

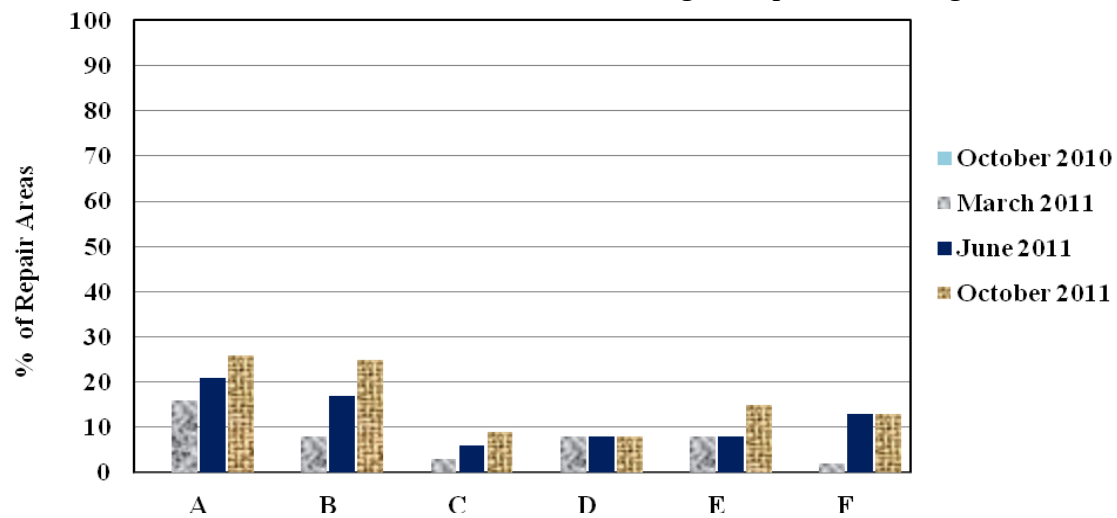


Figure 14: Percentage of Repair Areas with Spalls

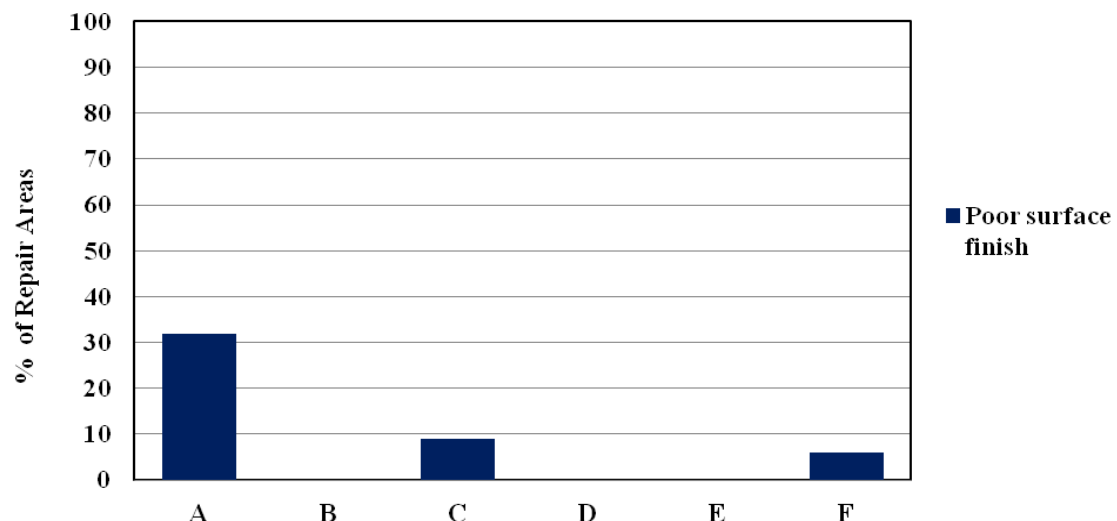


Figure 15: Percentage of Repair Areas with Poor Surface Finish

Table 3: Compressive Strength of Repair Materials

Repair Product	Compressive Strength [MPa]				Coefficient of variation (%)
	Sample 1	Sample 2	Sample 3	Average	
A	57.9	59.0	42.5 ^a	58.5	1.33
B	51.2	60.8	60.9	57.6	9.70
C	N/A ^b	N/A	N/A	N/A	N/A
D	52.7	41.5	49.5	47.9	12.04
E	34.5*	68.4	59.7	64.1	9.60
F	90.1	84.4	86.9	87.1	3.27

^a Compressive strength value was arbitrarily not included in average and standard deviation calculation

^b Data are not available

The bond strength between repair material and concrete was tested under compression failure using a test procedure adapted from ASTM C 882: *Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear* (ASTM 2008). Bond strength tests were conducted on 3×6 inches composite cylinders. Each sample was cast in two stages, first while the mold was slanted at 30 degrees, concrete was poured to fill one-half of the mold and the concrete was allowed to harden. Then, the sample was placed in an upright position and filled with one of the repair materials. The resulting cylinders had a concrete-repair material interface slanted at 30 degrees from the loading axis as shown in Figure 16. The bond strength of repair materials was evaluated for unconditioned and conditioned samples subjected to freeze-thaw and wet-drying cycling.

For freeze-thaw durability, bond strength was tested after 16 and 33 cycles of freeze-thaw conditioning. Each freeze-thaw cycle takes 36 hours to complete: 12 hours at -40°C, 12 hours of cycling the temperature between -10°C and +10°C, and 12 hours at +40°C. An environmental chamber was used to subject the samples to freeze-thaw cycling. For wet-dry durability, bond strength was tested after 4, 8, and 14 cycles of wet-dry conditioning. Each wet-dry cycle takes 48 hours to complete: 5 hours placed in a water bath at room temperature, 42 hours placed in an oven at +50°C, and 1 hour placed at room temperature. Three replicates were tested at each step. Samples that exhibited strength variation greater than 30% were excluded from the calculating the average.

Table 4 shows the bond strength of unconditioned, freeze-thaw conditioned, and wet-dry conditioned samples. The bond strength for all the repair materials ranged from 18.5 MPa to 22.2 MPa, except material A which had a bond strength of 13.0 MPa. Freeze-thaw and wet-drying conditioning did not show a significant drop in the bond strength between repair materials and concrete. Materials A, D, and F showed some increase in the bond strength after conditioning. The increase in the bond strength can be due to the continuing hydration of the cementitious materials in the repair product.

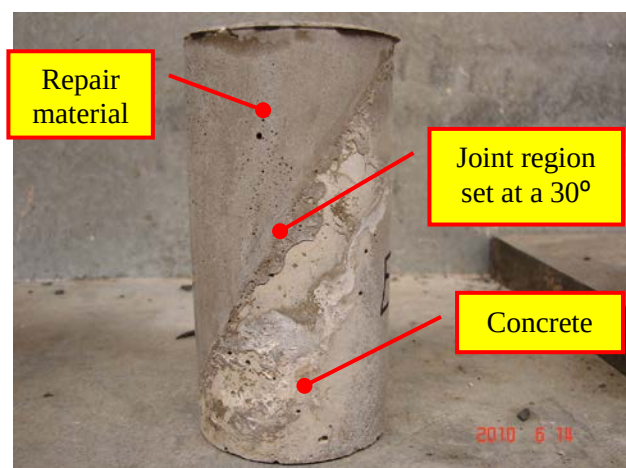


Figure 16: Composite Cylinder for Bond Strength Test

Table 4: Bond Strength between Repair Materials and Concrete

Repair Product	Unconditioned [MPa]	Freeze-Thaw Conditioned [MPa]		Wet-Dry Conditioned [MPa]		
		16 cycles	33 cycles	4 cycles	8 cycles	14 cycles
A	13.0	11.6 ^a	11.7	19.9	22.5 ^a	20.9
B	22.2	18.8	19.7	22.6	22.5	18.95
C	18.5	19.7	19.7	20.3	20.3	21.55
D	20.1	21.3	20.8	23.9	24.7	26.1
E	20.3	17.1	19.9	21.0	21.3	20.2
F	18.9	22.3	23.5	22.6	22.6	20.0

^a Bond strength is based on the testing of one sample. The second and third replicates failed before testing

Summary and Conclusions

The field and laboratory performance of six cementitious repair materials was investigated in this research. Two test sections were selected on a major arterial in the City of Winnipeg, Manitoba to compare the six candidate materials under the same environmental conditions and similar traffic loading. The field repairs were completed in September 2010. The performance of the repair materials were evaluated one, six, nine, and thirteen months after installation. The criteria that were used to evaluate the performance of the repair materials are: surface finish, transverse (shrinkage) cracking, longitudinal cracking, cracking at the saw-cut surface, surface wear, and spalls.

All the repair materials had good surface finish except material A. Areas repaired with materials E and F had lower percentage of transverse cracking than the remaining materials. Almost all the transverse cracking was observed one month after installation. Repair materials that had high water content during mixing experienced higher percentage of transverse cracking. The percentage of longitudinal cracking was very small for all repair materials except material B; this cracking can be attributed to the structural instability of the patched area and are not related to the type of repair material.

Materials D and F had the lowest percentage of cracking at saw-cut surface after one month, while material B the highest percentage of cracking. After thirteen months, material D was the only material that maintained low percentage of cracking at saw-cut surface. Signs of surface wear were observed for 33% of the patches repaired with material D after six months; this percentage ranged from 62% to 89% for the remaining repair materials. After thirteen months, all the repaired areas showed signs of surface wear. The percentage of patches with spalls ranged from 2% to 16% after six months. After thirteen months, the percentage of patches with spalls ranged from 8% to 26%.

The laboratory evaluation of the repair materials included evaluation of compressive strength, wet-dry durability of bond strength, and freeze-thaw durability of bond strength. The bond strength for all the repair materials ranged from 18.5 MPa to 22.2 MPa, except material A which had a bond strength of 13.0 MPa. Freeze-thaw and wet-drying conditioning did not show a significant drop in the bond strength between repair materials and concrete. Materials A, D, and F showed some increase in the bond strength after conditioning. The increase in the bond strength can be

attributed to the continuing hydration of the cementitious materials in the repair product. Similar findings are reported in the literature for cementitious repair materials subjected to thermal cycling (Al-Ostaz et al. 2010). The final evaluation results will be reported by the summer of 2012.

Sensitivity of the repair materials to the amount of water added during mixing was the main factor that affected the performance of the material. Result of the field surveys showed signs of degradation of the repair materials after the first winter. Further investigation need to be conducted to study the frost resistance of the repair materials and the effect of de-icing salt on the repair materials.

References

Al-Ostaz, A.; Irshidat, M.; Tenkhoff, B.; and Ponnappalli, P.S. (2010) "Deterioration of Bond Integrity between Repair Material and Concrete due to Thermal and Mechanical Incompatibilities", *Journal of Materials in Civil Engineering, American Society of Civil Engineers (ASCE)*, Vol. 22, No. 2, p 136-144.

American Concrete Pavement Association (2008) "*Concrete Pavement Field Reference: Preservation and Repair*".

American Society for Testing and Materials (2008) "*2008 Annual Book of ASTM Standards*"

Markey, S.M.; Lee, S.I.; Mukhopadhyay, D.G.; Whitney, Z.D.; and Fowler, D.W. (2005) "Investigation of Spall Repair Materials for Concrete Pavements", *Report nr FHWA/TX-06/0-5110-1*, Texas Department of Transportation Research and Technology Implementation Office, Federal Highway Administration, U.S. Department of Transportation.

Pattnaik, R., and Rangaraju, P. (2007) "Analysis of Compatibility between Repair Material and Substrate Concrete Using Simple Beam and Third Point Loading" *Journal of Materials in Civil Engineering, American Society of Civil Engineers (ASCE)*, Vol. 19, No. 12, p 1060-1069.

Soliman, H., Lambert, J-L., Shalaby, A., Liske, T., Kass, S., and Deane, L. (2011) "Characterizing Field and Laboratory Performance of Cementitious Partial Depth Repair Materials" *Presented in the Annual Conference of the Transportation Association of Canada*, Edmonton, Alberta.

Wilson, T.P.; Smith, K.L.; and Romine, A.R. (1999a) "Materials and Procedures for Rapid Repair of Partial-Depth Spalls in Concrete Pavements - Manual of Practice", *Report nr FHWA-RD-99-152*, Federal Highway Administration, U.S. Department of Transportation.

Wilson, T.P.; Smith, K.L.; and Romine, A.R. (1999b) "LTPP Pavement Maintenance Materials: PCC Partial-Depth Spall Repair, Final Report", *Report nr FHWA-RD-99-153*, Federal Highway Administration, U.S. Department of Transportation.