Innovative Preservation Treatments to Address Premature Pavement Roughness on a Swelling Clay Subgrade in Alberta

Mohammad Karim, M.A.Sc., P.Eng.
Sr. Pavements Engineer
EBA Engineering Consultants Ltd.
14940 - 123 Avenue, Edmonton, Alberta, T5X 5Z4 Canada
Email: mkarim@eba.ca
Tel: 780.451.2121

Dave Palsat, M.Sc., P.Eng.
Sr. Vice President
EBA Engineering Consultants Ltd.
14940 - 123 Avenue, Edmonton, Alberta, T5X 5Z4 Canada
Email: dpalsat@eba.ca
Tel: 780.451.2121

Art Johnston, C.E.T.
Principal, Transportation
EBA Engineering Consultants Ltd.
#115, 200 Rivercrest Drive SE, Calgary, Alberta, T2C 2X5 Canada
Email: ajohnston@eba.ca
Tel: 403.203.3355

Trent McLaughlin, P.Eng.
Construction Manager, Alberta Transportation
3rd fl Provincial Building, 9621 - 96 Avenue, Peace River, Alberta, T8S 1T4 Canada
E-mail: trent.mclaughlin@gov.ab.ca
Tel: 780.624.6366

Paper prepared for presentation
at the Pavement Evaluation, Performance, and Management Session
of the 2010 Annual Conference of the
Transportation Association of Canada
Halifax, Nova Scotia

ABSTRACT

In the early 2000s, Alberta Transportation (AT) twinned a portion of Highway 43 east of Grande Prairie in a staged sequence. The new westbound lanes exhibited premature roughness shortly after final paving. The observed roughness so early in the life of the pavement was extremely unusual and unexpected. Detailed investigation was carried out to determine the factors that were causing the premature roughness and to identify potential treatments that could be installed to restore ride quality or mitigate the recurrence of similar roughness. The assessment included a review of the as-built and historical information, a field trenching and drilling and a laboratory testing program, and field observations under winter and summer conditions. Profile data of the outer travel lane was collected during the summer/fall season of 2003 through 2008. In addition, data was collected in the winter of 2008 to assess seasonal effects and on the shoulder to assess the difference in roughness between the travel lanes and the shoulders. The analyses indicated that the roughness was characterized by heaving of the pavement surface in the vicinity of thermal induced cracks overlying a clay subgrade material with a high swell potential. The IRI has increased every year, and further increased in the winter. The road shoulders were smoother than the driving lanes. A mechanism that provided an explanation for the observed roughness was developed. Seven different treatment alternatives with different pre-overlay crack repairs were installed in 2009. This paper describes the comprehensive assessment activities, development of innovative treatments, and early performance.

INTRODUCTION

Several twinning projects constructed in the early 2000s in the Grande Prairie, Alberta area have been identified as exhibiting premature roughness [1]. Generally, these projects were granular based and first stage paved in 2000 or 2001 and final stage paved in 2002 or 2003. Highway 43 (Hwy 43) is one of these projects. Hwy 43 is part of Alberta's North South Trade Corridor that connects Alberta with the U.S. at Coutts and extends to the B.C. border west of Grande Prairie, going to Alaska. The new westbound lanes exhibited premature roughness shortly after final staged paving [1]. The observed roughness so early in the life of the pavement was extremely unusual and unexpected.

Roughness is defined as "a distortion of the pavement surface that contributes to an undesirable or uncomfortable ride" [2]. International Roughness Index (IRI) has become a standard for pavement roughness measurements [3]. An IRI value of 0 mm/m indicates absolute smoothness and an IRI value in the order of 10 mm/m represents a rough unpaved roadway [3]. AT also uses IRI as a performance measure on its entire road network. The average IRI value to trigger rehabilitation on this highway, for the volume of traffic it carries, is 2.3 mm/m [4]. There were sections of Hwy 43 that exhibited IRI of up to 2.9 mm/m in August 2008. This roughness appeared to be characterized by heaving of the pavement surface in the vicinity of transverse crack locations. It was reported that the ride quality had deteriorated with time and may be more severe in the winter. The roughness was reported to be most severe on the Hwy 43:04 WB project and has received public and media attention.

It has been reported that traffic has been observed driving the shoulders to avoid the heaves that appeared to be more dominant in the travel lanes (Photo 1). AT identified the need for an engineering study to investigate the performance of these projects. This paper summarizes the methodology to investigate the unexpected roughness, determine the factors causing the roughness, and identify potential repair and or preservation treatments that could be installed to restore the ride quality or possibly mitigate the recurrence of similar roughness distresses.

Photo 1: A truck driving on outside shoulder of Hwy 43:04 WBL

REVIEW OF BACKGROUND INFORMATION

Hwy 43:04 WBL km 0.00 to km 2.00 was constructed unstaged in 2002. Km 2.00 to km 28.98 were based and first stage paved in 2000/2001 and final paved in 2003. According to AT's Pavement Management System (PMS), as-built cross-section records, and review of the summary reports from the time of construction, the existing pavement structure consisted of 350 mm to 450 mm Granular Base Course (GBC) and 180 mm to 240 mm Asphalt Concrete (AC) in the outer lane; and 300 to 450 mm GBC and 180 to 220 mm AC in the inner lane. A review of a pre-design preliminary geotechnical investigation report [5] indicated that the road is built through flat lacustrine plain (this was also confirmed by reviewing the Surficial Deposits Map of the area [6]). The clay was reported to be high plastic with Liquid and Plastic Limit values of 53 to 65 percent and 20 to 24 percent, respectively. The report also found that these soils have low potential for frost action and moderate susceptibility for swelling. Some soils samples encountered had high potential for frost action. The report indicated that some moisture conditioning to bring the soils closer to optimum moisture content would be required for compaction.

Daily Compaction Reports of the grading and subgrade work indicated that the Optimum Moisture Content (OMC) of the clay subgrade referenced during construction compaction testing were between 22 and 24 percent. Construction records show that at the time of construction, the moisture content of the subgrade typically varied from 18 to 20 percent, indicating that many areas of the subgrade may have been constructed up to 4 percent dry of the OMC. Some areas were reported even "too dry". Figure 1 provides a scatter chart

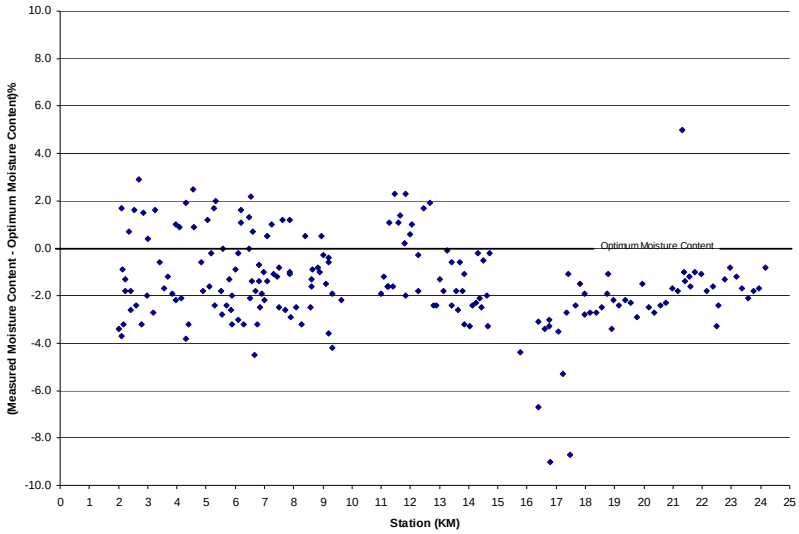


Figure 1: Quality Assurance Test results of Moisture Contents at the time of construction

of the measured moisture contents plus or minus the reported OMC at different locations. The Surfacing Strategy Report for final paving [7] indicated widely spaced transverse cracks between km 0 and km 5.5 and "odd" (random) transverse cracks from km 5.5 to the end of the project. These observations indicated that low temperature transverse cracks were apparent in the first stage paving. Asphalt test results indicated that the asphalt used on this roadway for final paving was characterized as PG 58-29.

HISTORIC / SEASONAL EFFECTS ON INTERNATIONAL ROUGHNESS INDEX

The International Roughness Index (IRI) and profile data was collected on this highway in the driving lanes and on outer shoulder between 2003 and 2009. The WBOL/shoulder data was collected with the survey vehicle positioned with the driver's side wheel in the OL outer wheelpath and the passenger side wheel in the mid-shoulder.

The section of Hwy 43:04 WBOL from km 0.000 to km 28.984 was segmented into relatively homogeneous sections based on historical IRI values. Figure 2 presents the section average IRI values for the outer lane outer wheelpath for surveys carried out from October 2003 to March 2009. The following observations are provided:

- Average as-built IRI in 2003 following final paving was ~0.7 mm/m.
- There has been a steady increase in IRI ranging from about 0.2 to 0.7 mm/m per year.
- From August 2007 to March 2008, section average IRI values increased about 0.3 to 0.8 mm/m.
- From March 2008 to August 2008, section average IRI values decreased about 0.2 to 0.6 mm/m.
- The increase in March 2008 from August 2007, and the subsequent decrease by August 2008, confirms winter freezing/frost is affecting IRI.
- There was a substantial increase in IRI from March 2008 to March 2009 in all cases.

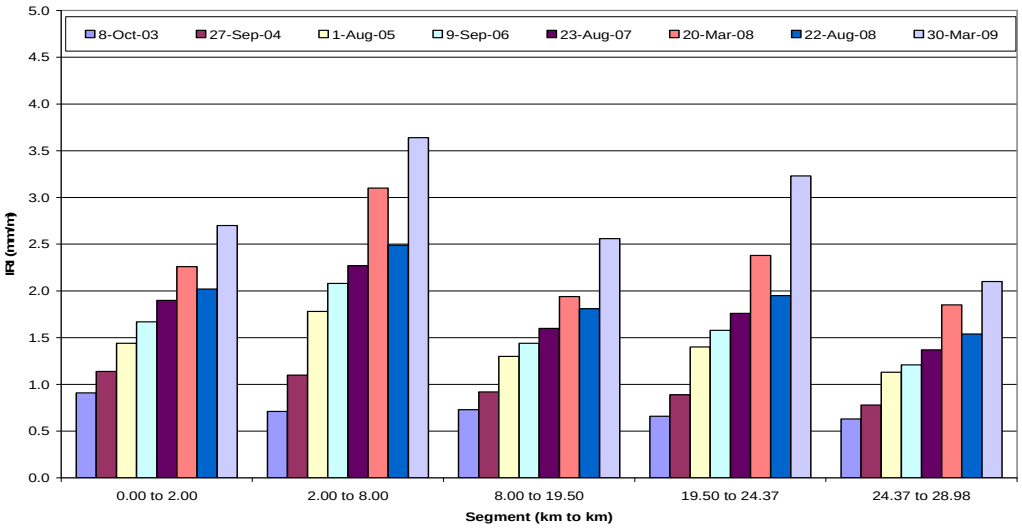


Figure 2: Hwy 43:04 WB outer lane outer wheelpath - progression of IRI from 2003 to 2009

During a field reconnaissance carried out in March 2008, it was observed that smoother sections appeared to be associated with a lesser frequency of transverse cracks. The frequency of full width transverse cracks was determined for each km. Figure 3 presents the August 2008 1 km average IRI for the outer lane and the frequency of transverse cracks per km. Figure 4 provides a scatter chart comparing the two data sets.

A review of Figures 3 and 4 indicates that, in general, there is a strong correlation between 1 km average IRI and 1 km transverse crack frequency and would suggest that a significant proportion of the roughness is associated with the observed heaving at transverse cracks: the higher the frequency of transverse cracks, the rougher the pavement.

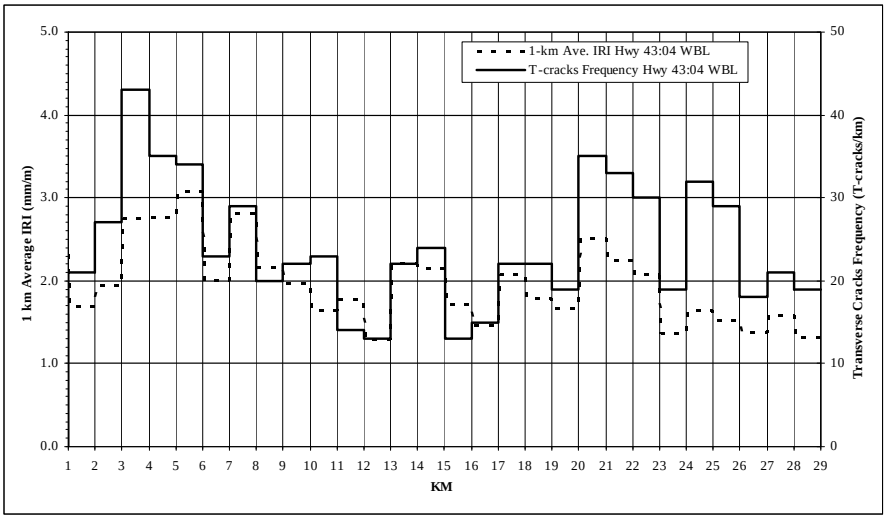


Figure 3: Hwy 43:04 Westbound Outer Lane August 2008 IRI and Transverse Crack Frequency

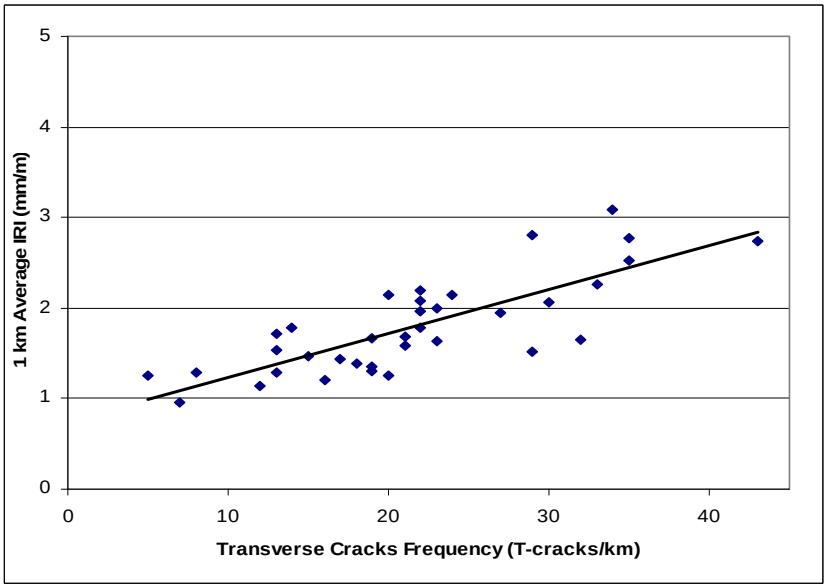


Figure 4: Hwy 43:04 Westbound Outer Lane – comparison of August 2008 1 km IRI and Transverse Crack Frequency

TEMPORAL AND SEASONAL EFFECTS ON ROADWAY PROFILE

The analysis of roadway profile data collected and archived by EBA on all surveys (except for 2007) provided an opportunity to study the extent and severity of heaving (both longitudinally and transversely), the change in heave profile with time, and the effect of winter freezing/frost on heave profile. A detailed analysis of both longitudinal and transverse profile data was carried out at many locations. For the purposes of this paper, one location is presented for the longitudinal profile analysis and one location for the transverse profile analysis. It should be noted that the analyses for all the analyzed locations presented similar results.

Analysis of Longitudinal Profile

One of the laser-based high-speed inertial profilers owned by EBA was employed for all surveys except for 2007. The heart of this system is an International Cybernetics Corporation (ICC) MDR4087 Road Profiler, considered a South Dakota DOT style FHWA Class II road profiler. All data were collected in accordance with AT requirements. The road profiler/rut configuration incorporated 11 infrared laser sensors and two high precision accelerometers located in each wheelpath. The overall width of the profiler/rut bar was 2.9 m. The high sample rate laser sensors in each wheelpath allows profile points to be pre-processed in hardware and stored at intervals as small as 19 mm at 110 kph. The combination of reduced sample interval and high quality accelerometers allows the system to accurately record surface undulation wavelengths from less than 0.3 m to more than 91 m, as required by current International Roughness Index (IRI) algorithms.

Hwy 43:04 westbound outer lane km 7.36 is located within the section of Hwy 43:04 based/paved in 2000 and final paved in 2003. Figure 5 provides the historical inside wheelpath profiles for the WB outer lane between km 7.3 and 7.4.

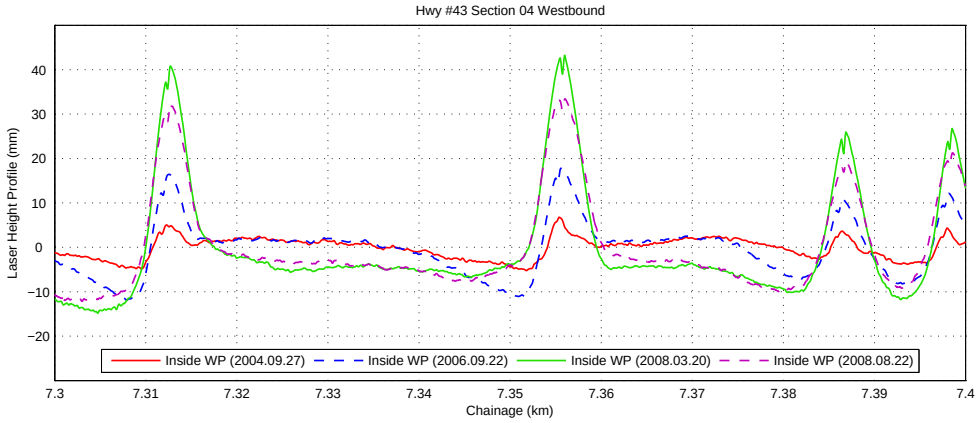


Figure 5: Hwy 43:04 WBL – change in WB outer lane IWP profile from 2004 to 2008

From the September 2004, September 2006, March 2008, and August 2008 surveys, the following observations are provided:

- Four distinct heaves can be observed that correspond to transverse crack locations at about km 7.312, 7.356, 7.386, and 7.398.
- There is evidence of a heave of about 5 mm in September 2004, one year following final paving and four years following first stage paving.
- The heaves have increased reaching a maximum summer value in August 2008 of about 30 to 35 mm.

Figure 6 provides the inside and outside wheelpath profiles for the WB outer lane for the March 2008 and August 2008 surveys from km 7.3 to km 7.4. The following observations are provided.

- Four distinct heaves can be observed that correspond to transverse crack locations.
- The heaves appear to have subsided about 5 to 10 mm from March 2008 to August 2008.

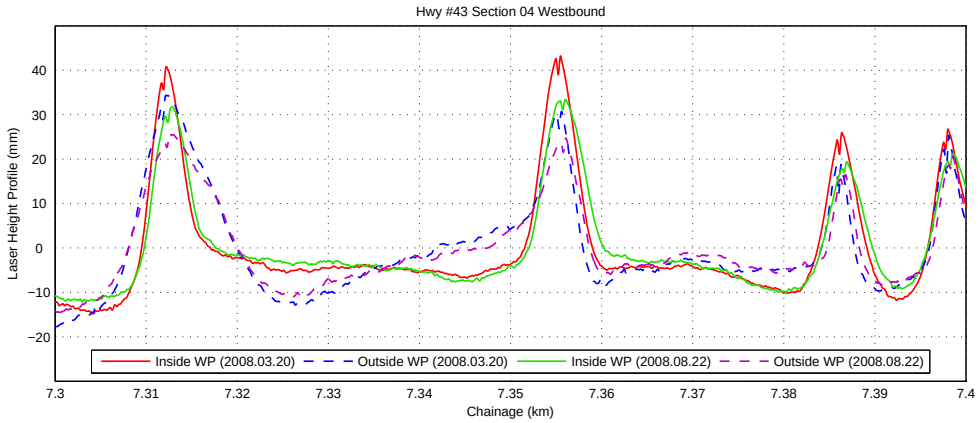


Figure 6: Hwy 43:04 WBL – change in WB outer lane IWP and OWP profiles from March 2008 to August 2008

Field observations and limited measurements carried out on site in March 2008, indicated that the heaves were all located at transverse crack locations, were greatest at the centre line of the road, diminishing transversely to virtually zero at the shoulder edge, and diminishing longitudinally over several metres in both directions from the transverse crack. Analyses of the longitudinal profile for the inside wheel path, outside wheelpath, and mid-shoulder for the WB outer lane between km 7.30 and km 7.40 from March 2008 and August 2008 indicated:

- The heave height in March 2008 was generally greatest in the inner wheelpath location reducing 5 to 10 mm in the outer wheelpath and further reducing at mid-shoulder.
- The heave heights in August 2008 follow the same pattern as for March 2008, but were generally reduced by 5 to 10 mm at each location.

Analysis of Transverse Profiles and Cross Slope Profiles

A limited study was carried out to evaluate the transverse profile or cross slope at many locations using inertial profile data collected during the August 2008 survey. One location was selected for

presentation here. The cross slope was calculated for both the right half and left half of the rut bar separately, i.e., over a width of about 1.45 m. Figure 7 provides the five calculated cross slopes between km 25.4 and km 25.5, horizontal curve and has a superelevated cross-section with the cross-section sloping from the median to the outside shoulder (note that the "noise" associated with the outside shoulder cross slope is due to the rumble strip installation). The as-built cross slope was not available; based on a radius of 2,400 m (from data available at AT's website) and design speeds of 120 km/h and 130 km/h, the superelevation based on AT design criteria would be 3.0 percent and 3.4 percent, respectively.

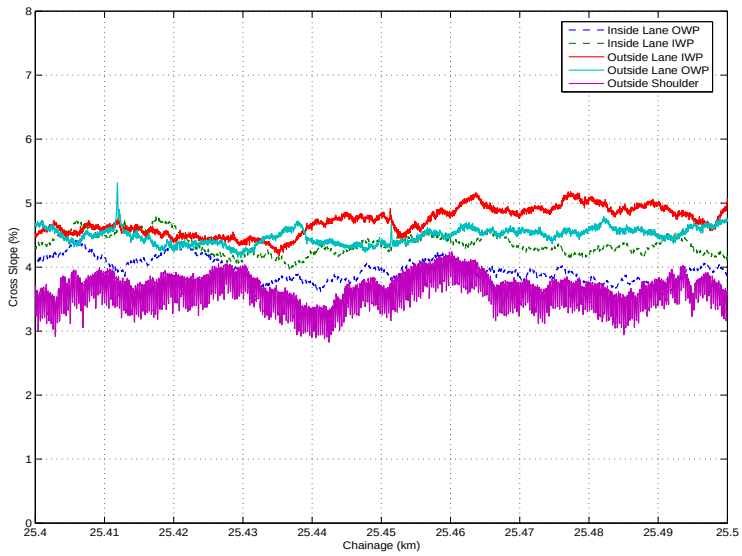


Figure 7: Hwy 43:04 WBOL and WBIL Horizontal Curve at km 25.5

The following observations are provided:

- The outside shoulder cross slope is the flattest of all cross slopes and is between 3 and 4 percent.
- The cross slopes generally increase toward the upper side of the superelevation with the cross slope of the inside lane OWP and IWP being between 4.5 and 5 percent.

GEOTECHNICAL INVESTIGATION

Field trenching was conducted March 24 through 26, 2008 on the shoulder of Hwy 43:04 WBL at three selected locations. The primary intent of conducting the trenching program during the winter was to see if there were ice lenses forming in the granular base or subgrade that would cause frost heaving. The trenching was carried out on the shoulders for safety reasons although the heaves were much less than the travel lanes. Subsequent to the trenching program, a drilling program of the lanes was carried out April 4 through 6, 2008. The drilling was carried out at the time when frost was still in the ground to see if any potential frost actions were contributing to the extreme heaving. The drilling program was conducted with solid stem auger techniques and included coring with a specialized core barrel designed to core frozen soil to collect undisturbed soil samples towards the

centreline of the road, as well as in the outer wheelpath close to the transverse crack and away from the area influenced by heaving. A total of eight borehole locations were selected at two of the three trenched sites. Undisturbed core soil samples from all the eight locations were collected, logged, photographed, and delivered to EBA's Edmonton laboratory for testing. This paper details the geotechnical investigation of only one site. It should be noted that the results of all three test sites were very similar.

Test Site 1 was selected at a transverse crack location at km 7.360. A trench was excavated on the outer shoulder. The trench was about 4.0 m east and 1.0 m west of the crack and about 2.0 m deep from the surface of the pavement. Core holes were excavated: two at the crack location where the heave was maximum, and two away from the influence of heaved area. Locations of the trench and core holes are shown in Photo 2. No ice lensing, except for some small ice crystals, was observed anywhere in the trench. Transverse cracks were not observed to extend through GBC layer.

Photo 2: Looking west at Test Site 1

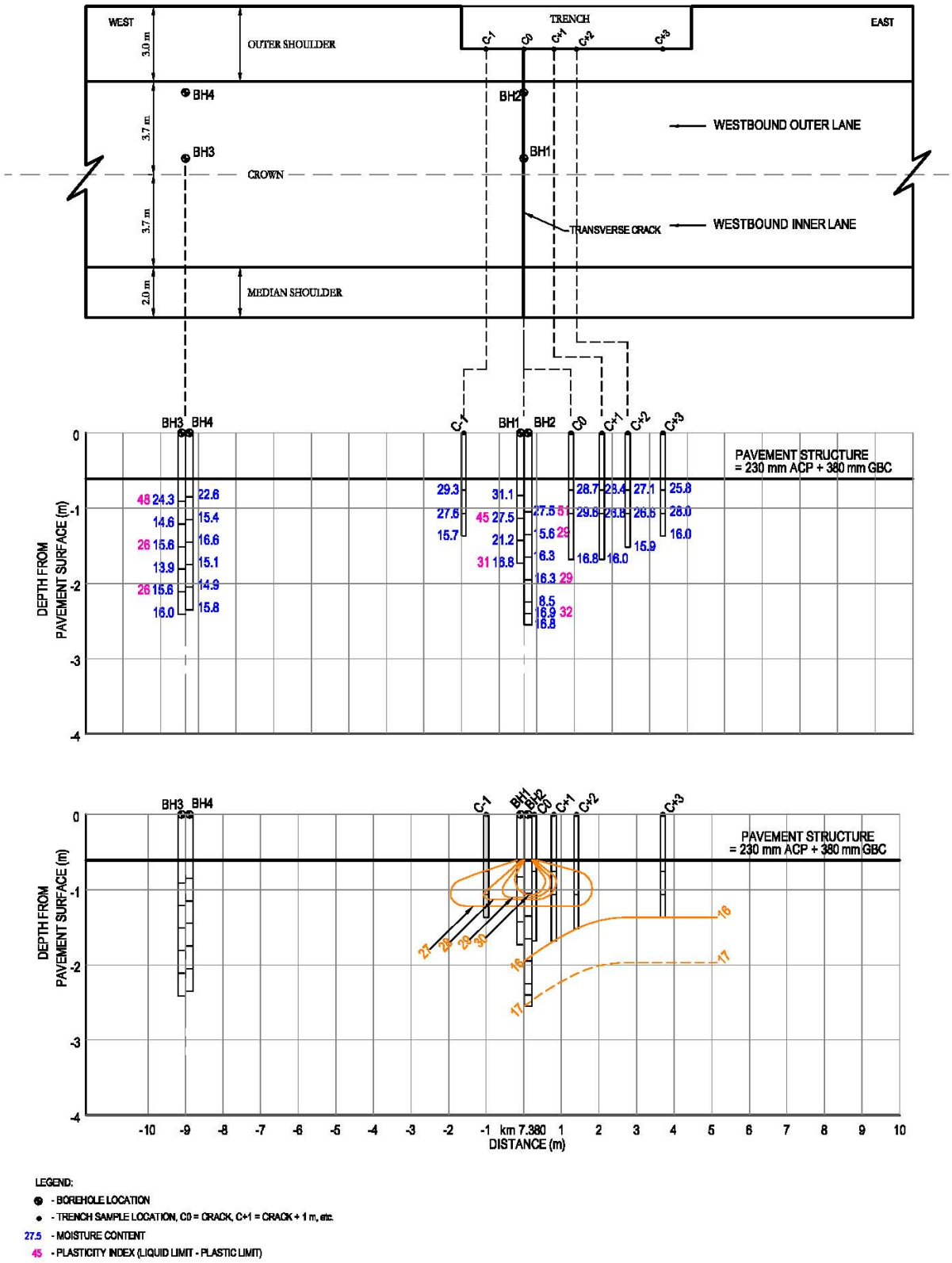
Soil samples were retrieved from the trench, as well as from the core holes at the locations shown in Figure 8. Segregated visible ice commonly associated with frost heaving was not observed during EBA's subsurface investigation at this site. Soil temperature was measured at different depths of the trench and is reported in Figure 8. All samples were visually classified in the field and the individual soil strata and the interfaces between them were noted. Results of the moisture content and Plasticity Indices (PI) for the soil samples are presented in Figure 8. Plastic Limits ranged from 16 to 24 percent and Liquid Limits ranged from 42 to 72 percent with PI from 26 to 51 percent. Contour maps of the moisture contents are drawn on Figure 8, which shows that the moisture content decreases gradually away from the crack and with depth. For the Test Site 2 at km 9.38, Plastic Limits ranged from 22 to 23 percent and Liquid Limits ranged from 68 to 77 percent with PI from 46 to 55 percent. For the Test Site 3 at km 20.94, Plastic Limits ranged from 16 to 24 percent and Liquid Limits ranged from 35 to 84 percent with PI from 28 to 55 percent.

DISCUSSION ON CLAY SUBGRADE MATERIALS

The observations and data summarized in the preceding sections indicated that two mechanisms were causing the heave of the asphalt pavement:

- Swelling of unsaturated high plastic clay.
- Frost action of clay that has absorbed water to satisfy internal suction pressures.

Each mechanism is described in the following sections.



LEGEND:

- BOREHOLE LOCATION
- TRENCH SAMPLE LOCATION, C0 = CRACK, C+1 = CRACK + 1 m, etc.

27.5 - MOISTURE CONTENT

45 - PLASTICITY INDEX (LIQUID LIMIT - PLASTIC LIMIT)

Figure 8: Test Site 1, Locations of Soil Samples and their Test Results

Swelling Soils

Swelling in soil is a complex process that depends on the clay minerals present in the soil, the soil structure, the degree of saturation, and several physico-chemical aspects of the soil [8]. The natural soil subgrade and the common fill/borrow materials used to construct the highway embankment have either medium or high plasticity. Such soils have a tendency to swell if they are compacted at moisture contents less than the optimum moisture content determined through moisture-density relationship testing, and when exposed to sources of water. Sources of water in a highway embankment can come from surface water infiltration through cracks in the pavement or capillary action, with water being drawn from wetter areas to drier areas. In this case, the water is entering the high plastic clay subgrade through two general avenues: through the transverse cracks, and at the edge of the pavement and through the shoulder of the fill from the ditches.

Due to the low permeability of the high plastic clay subgrade, groundwater at depth is not expected to contribute as much water as the two aforementioned sources. The pronounced heave at the transverse cracks is due to the ingress of water into the cracks. The contours of moisture content presented in Figure 8 show the results of this effect. Although the heave appears to be less pronounced at the shoulders, the IRI data shows that the shoulders in general have heaved to create flatter outer cross slopes than normally used in highway design. This has tended to mask the heave at the shoulders at each transverse crack location.

Overall, the relatively dry as-built moisture condition of the clay subgrade was causing the majority of the heave. When used as construction materials, medium and high plastic soils should be compacted 1 to 2 percent wet of optimum moisture content to increase the degree of saturation that correspondingly reduces the suction pressures within the soil that can cause water to be drawn into the soil.

Frost Heave

The seasonal change in the amount of heave measured by the profile and IRI data indicates that frost heaving is a factor that is resulting in higher heaves and higher IRI values in the winter than in the adjoining summer seasons. Frost heave is generally due to two factors:

- 9 to 12 percent expansion of water upon freezing.
- Development of ice lensing in response to a supply of water just below the advancing frost front.

Water expands by 9 percent during the freezing process due to the development of a lattice structure of water molecules in its solid (i.e., ice) form that has a lower unit weight than in its liquid form. However, unsaturated soil conditions have inherent air filled voids that can accommodate some of the expansion of the water in the pores that tends to subdue the amount of heave experienced by the soil mass. More saturated soil conditions, such as those immediately below the transverse cracks

would tend to display more heave in response to freezing. This would create a relatively mild differential frost heave, which is reflected in the IRI data.

Ice lensing generally occurs at or just above the freezing front or 0° isotherm. The conversion of liquid water to a solid can establish high suction gradients that can draw water towards the freezing. However, high plastic soils typically have low permeability that restricts the amount of water that can be drawn towards the freezing front. Frost heave potential is controlled by the structure of the soil and particle size distribution. The frost heave for re-worked high plastic clay is generally low partially due to destruction of any structure that would allow for preferential water migration to the freezing front and low permeability that restricts the amount of water that can then be drawn towards the freezing front. Therefore, ice lensing in high plastic clay fill in northern Alberta would not be expected.

DISCUSSION ON ACP LOW TEMPERATURE PROPERTIES

Given the nature of the observed distress and the resulting performance deficiencies, low temperature induced transverse cracking was considered as a contributing factor. As such, a review of the subject project with respect to low temperature properties of the ACP has been undertaken. Based on the available records for the westbound lanes of Hwy 43:04, it has been confirmed that a 150/200A asphalt cement was used for the entire ACP thickness from km 0.0 to km 2.0 (constructed in 2002), and the final stage paving from km 2.0 to km 28.98 (constructed in 2003). Although it has not been confirmed that 150/200A asphalt cement was used in the mixes used for first stage paving between km 2.0 and km 28.98 in 2000 and 2001, it has been assumed to be the case for the purposes of this discussion.

Significant with respect to the binder properties for this project is that two samples, one from the 2002 construction and one from the 2003 construction, where characterized using Superpave protocols. In 2002, the sample tested indicated an actual Performance Grade (PG) of PG 58-29. In 2003, the indicated binder gradation was PG 57-29. Based on the closest weather database, Grande Prairie Airport, and using the Transportation Association of Canada (TAC) protocols for low pavement temperature, the Design Low Temperature associated with a 50 percent reliability is -31.4°C. Therefore, based on the limited test data characterization, the asphalt binder used on Hwy 43:04 would provide less than a 50 percent reliability of no low temperature cracking (i.e., theoretically there would be over a 50 percent chance that low temperature induced cracking would occur in any given year). Figure 9 presents the Grande Prairie Airport low temperature data for the winter periods (December to March, inclusive) for 2000/2001 to 2005/2006. Also shown are the Low Design Temperatures corresponding to 50, 85, and 98 percent reliability. It should be recognized that the temperature data presented is minimum air temperature and the design low temperature values are pavement temperature. Pavement temperatures are somewhat warmer than air temperature (typically 3° to 5°C, the difference increasing with lower temperature), and this should be considered when interpreting the data.

Several observations can be made with respect to the data presented in Figure 9.

- Based on a low temperature grading of -29°C, as indicated by the test data, the capability of the binder used on Hwy 43:04 to resist low temperature induced cracking, in terms of pavement temperature, was likely exceeded in four of the six winter seasons illustrated (recognizing that the ACP construction took place in 2000 through 2003).

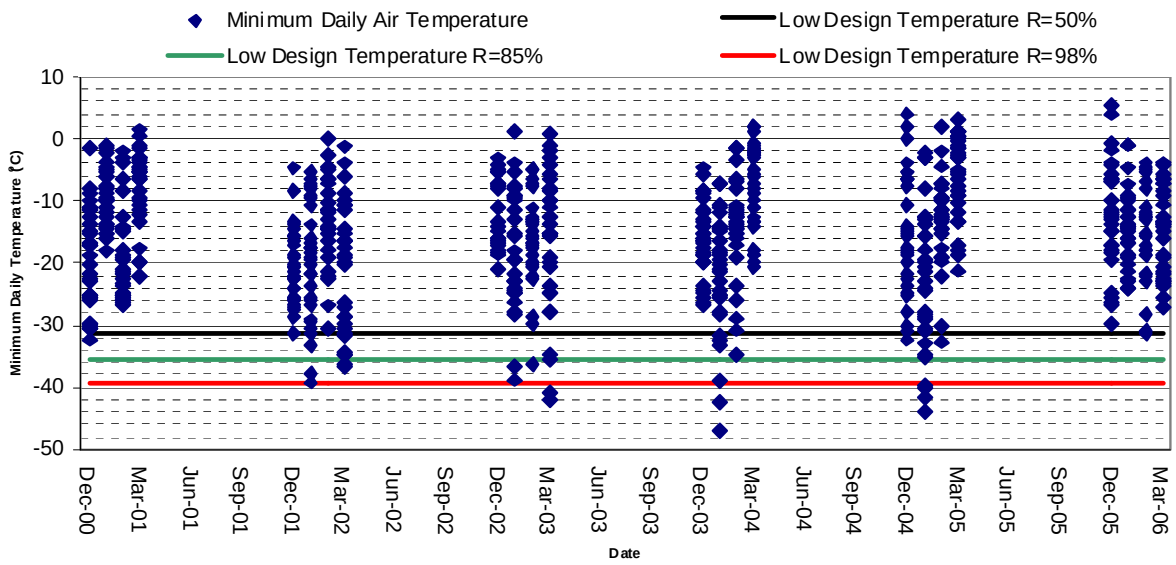


Figure 9: Historical Low Temperature Data (Grande Prairie Airport – Winter 00/01 to 05/06)

- Had a binder been used that satisfied 85 percent reliability, the low temperature capacity of that binder would likely have been exceeded in at least three of the six winters.
- Selection of a binder meeting 98 percent reliability criteria may have still been susceptible to low temperature induced cracking during the winter of 2003/2004.

Based on the low temperature properties of the asphalt binder used in the mixes for Hwy 43:04, the review indicated that low temperature induced cracking of the subject pavement was inevitable. Based on the examination of the periods encompassing the ACP construction, the potential for low temperature cracking was evident in four of the six winters. A binder grade of PG XX-40 would be expected to provide a 99 percent reliability of low temperature induced cracking mitigation. Assuming that a high temperature grade of 58°C is warranted for high temperature performance, a PG 58-40 would be necessary to significantly improve low temperature performance in this climate.

SUMMARY OF OBSERVATIONS FROM THE ANALYSES

Based on the analyses and evaluation presented, key observations are summarized below:

- There has been a significant and continuing yearly increase in summer IRI values since construction of final paving.

- There is a strong correlation between 1 km average IRI and 1 km transverse crack frequency. In general, the higher the frequency of transverse cracks, the rougher the pavement.
- Low temperature transverse cracking was first observed in the first stage pavement one year following construction.
- Heaves at transverse crack locations first appeared one year following final paving.
- The heaves are growing in height with time.
- As would be expected, the heaves were greater in March 2008 than in August 2008, indicating winter effect.
- The height of the heaves is generally greatest in the inner wheelpath location and diminishes towards the shoulder.
- The heave heights in the horizontal curves at km 25.5 and km 27.2 (which was constructed under a separate grading contract from the sections west of km 24.340 and was base/paved in 2001 and final paved in 2003) are significantly less than the heave heights on the tangent sections west of km 24.370.
- Based on the cross-slope analyses, the cross-slope values are lowest on the low side of superelevated curves, or on the outside shoulder of the tangent section, compared to the adjoining lanes.
- No frost lensing was observed in the subgrade at any of the trenches excavated at the three locations in early March 2008.
- The moisture content in the subgrade in the vicinity of transverse cracks was 7 to 10 percent higher than the surrounding subgrade.

MECHANISM THAT CAUSED THE OBSERVED DISTRESSES

A major objective of this study was to determine the factors that are causing or influencing the premature roughness. Based on evaluation of the available data, the following discussion provides a sequence of events and circumstances that could lead to the performance of this roadway.

1. High swell potential clay materials were used to construct the subgrade.
2. The clay subgrade was constructed at moisture contents less than the optimum moisture content, which created a condition where the fill was substantially unsaturated. This resulted in a latent defect in that the soil would heave wherever it was exposed to water.
3. The properties of the asphalt binder used in the first-stage paving, and very low winter air temperatures, resulted in the initiation of low temperature transverse cracks. These transverse cracks subsequently reflected through the final pavement layer. The properties of the asphalt binder used in the final paving, and very low winter air temperatures, resulted in the initiation of additional low temperature cracks.
4. Upon completion of first-stage paving, additional run-off water off the pavement surface enters the clay side slope, providing an additional source of water resulting in swelling of the side slope and subgrade under the shoulder, and heaving of the paved shoulder.
5. Following the initiation of transverse cracks surface water is forced under the pressure of tire dynamics, and enters the pavement and granular base. The permeability of the granular base course is not high enough to allow water captured at the top of the subgrade to immediately drain laterally to the side slope. The water "pools" within the granular base and enters the

underlying subgrade causing localized swelling and heaves of the pavement surface in the vicinity of the transverse cracks.

6. Areas with a higher frequency of transverse cracks result in a higher frequency of heaves and a higher IRI. (The higher frequency of transverse cracks could result from variations in as-supplied asphalt binder properties, variations in in-place binder properties due to plant mixing condition, localized variations in winter air temperatures, variations in ACP thickness, construction of un-recorded granular sub-base layers.)
7. The heaving at transverse crack locations increases in the winter due to the expansion of water in the subgrade upon freezing. Higher subgrade moisture contents in the vicinity of transverse cracks would result in greater heaving than areas away from the cracks where moisture contents are lower.
8. At many locations, the heaves at transverse cracks are greater towards the centre line and reduce towards the shoulder edge. This could be due to pre-heaving along the outside portions of the cross-section due to run-off water off the pavement surface and also because the subgrade near the shoulders can swell vertically and horizontally, whereas, the subgrade at centre line can only swell vertically.

REHABILITATION STRATEGIES

The preservation strategies described in Table 1 were identified by the design report [9]:

Table 1: Description of Preservation Treatments and Their Locations

Segment & Treatment	Location (km to km)	Description of the Treatment
1-A	km 0.00 to km 2.00	Cold mill (50 mm) and inlay driving lanes; re-profile transverse cracks on shoulders using fine grind; and fog shoulders
2-B	km 2.00 to km 3.00	Re-profile full surface by cold milling about 20 to 30 mm depth and 50 mm single lift overlay
3-C	km 3.00 to km 4.00	Saw cut the transverse crack locations about 0.6 to 1.2 m wide; remove and replace the existing GBC and ACP; re-profile the full width and install 50 mm single lift overlay; and saw cut (200 mm deep) and rout and seal at the old crack location
4-B	km 4.00 to km 8.00	Same as Treatment 2-B above
5-D	km 8.00 to km 11.00	Same as Treatment 1-A above + saw cut (50 mm deep) and rout and seal at the old transverse crack locations
6-E	km 11.00 to km 12.95	Mill and inlay (50 mm) transverse cracks full width and about 10 m in the driving direction
7-A	km 12.95 to km 18.10	Same as Treatment 1-A above
8-F	km 18.10 to km 20.10	Re-profile by fine grinding only at the transverse crack locations and 50 mm single lift full width overlay
9-G	km 20.10 to km 25.90	Mill and fill selected transverse cracks (400 mm wide and 50 mm deep); saw cut 50 mm deep and rout and seal; and 70 mm two lift overlay
10-E	km 25.90 to km 27.52	Same as Treatment 6-E above
11-H	km 27.52 to km 28.04	Re-profile transverse cracks about 10 m in the driving direction by fine grinding and seal with double slurry seal coat
12-E	km 28.04 to km 28.98	Same as Treatment 6-E and 10-E above

Details of cross-sections cannot be given in the paper due to limited space, but can be found in [9] or requested from one of the authors. The construction work was carried out in August and September of 2009. Observations and analysis of pre-construction and post-construction profile data over the first winter since construction is presented in the following sections. The long-term monitoring of the installed treatments is planned and will be reported in the future.

POST CONSTRUCTION IRI DATA ANALYSIS

The main objective of installing different treatment alternatives was to evaluate what effect each treatment has in addressing the premature roughness. One of the observations from previous analyses was that the roughness increased each year and further increased in winter. For this reason, the IRI was also collected in January 2010 following the construction. Based on the type of treatments installed, the project was divided into 12 segments. The following IRI and profile data collected by EBA's inertial profiler on Hwy 43:04 westbound outer and inner lanes (WBOL and WBIL) were analyzed:

- August 2008 The most recent pre-construction data available for summer/fall 2008
- March 2009 The most recent pre-construction data available for winter 2008/2009
- October 2009 The first post-construction data available for summer/fall 2009
- January 2010 The first winter data after construction 2009/2010

The data were collected in all four wheelpaths of the driving lanes. The same laser-based high speed inertial profiler was employed for all surveys. Figure 10 presents the decrease/increase in mean IRI values for all segments in the westbound outer lane (WBOL) from before construction both in winter and in summer. Figure 11 presents the difference in mean IRI in the WBOL between March 2009 and January 2010, and August 2008 and October 2009 for all 12 segments.

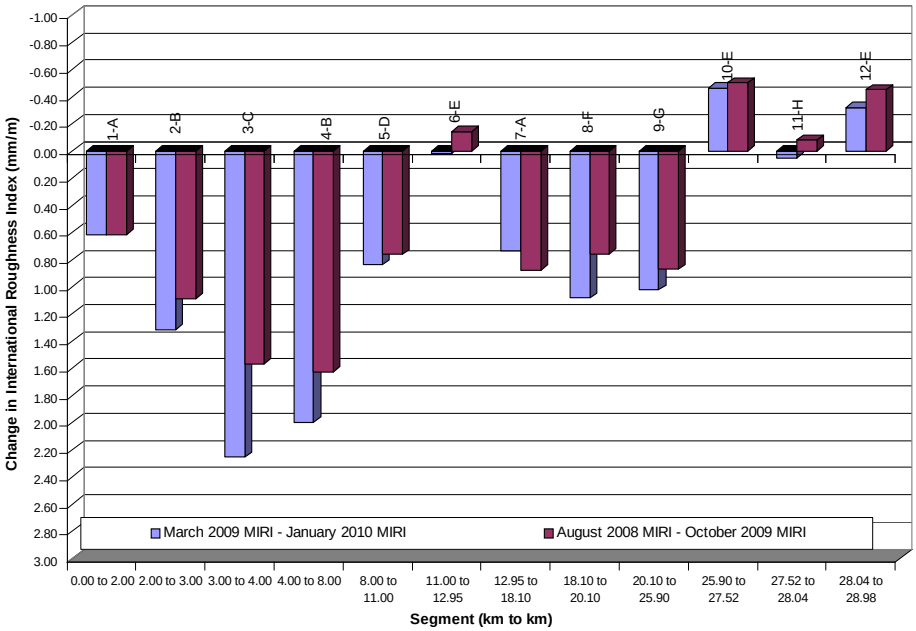


Figure 10: Hwy 43:04 WB outer lane – effect of each treatment on MIRI

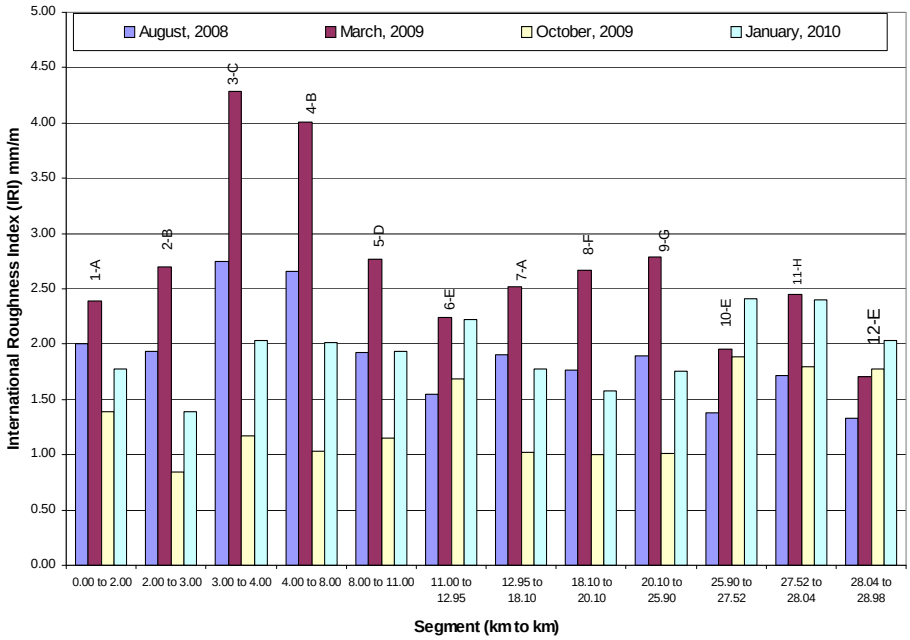


Figure 11: Hwy 43:04 WB outer lane – pattern of changes in IRI for different treatments

SUMMARY OF OBSERVATIONS FROM THE ANALYSIS

The performance of the westbound inner and outer lanes was very similar based on IRI. For this analysis, only the outer lane has been considered. Table 2 provides a summary of average MIRI data for the WBOL for each segment for all four surveys. Also presented is the difference between August 2008 MIRI and October 2009 MIRI, which represents the improvement in ride due to the installation of each treatment. The difference in MIRI between March 2009 and January 2010, which represents the improvement in winter ride over the last year, is also presented. The sections are presented in order from highest to lowest pre-construction (August 2008) MIRI. Based on the order of presentation from highest to lowest pre-construction MIRI, the segments have been separated into three groups. Observations from Table 2 and the field reconnaissance indicate:

Group 1 – Highest Pre-Construction MIRI

Segments 3-C and 4-B were the roughest of all sections prior to installation of the treatments and were improved the most (~1.6 mm/m reduction due to installation of the treatments and 2.0 to ~2.3 mm/m reduction in winter MIRI), irrespective of the type of pre-overlay crack repair. The field reconnaissance indicated that the segment 3-C where ACP and GBC at the transverse crack location was removed and replaced, and a crack was introduced by saw-cutting, did not present any heaving or dipping at that location. The edges of the repair in Segment 3-C had hairline cracking. Segment 4-B indicated that the transverse cracks were still slightly heaved in the winter of 2009/2010.

Table 2: Summary of Average MIRI Data for Westbound Outer Lane for Each Segment						
Segment & Treatment[1]	**August 2008 MIRI**	**October 2009, MIRI**	**August 2008 to October 2009 MIRI Change**	**March 2009 MIRI**	**January 2010 MIRI**	**March 2009 to January 2010 MIRI Change**
3-C	2.75	1.18	**1.57**	4.29	2.04	**2.25**
4-B	2.66	1.03	**1.63**	4.01	2.01	**2.00**
1-A	2.00	1.39	**0.61**	2.40	1.78	**0.62**
2-B	1.94	0.85	**1.09**	2.70	1.39	**1.31**
5-D	1.92	1.16	**0.76**	2.77	1.93	**0.84**
7-A	1.91	1.03	**0.88**	2.52	1.78	**0.74**
8-F	1.77	1.01	**0.76**	2.67	1.58	**1.09**
9-G	1.89	1.02	**0.87**	2.79	1.76	**1.03**
6-E	1.55	1.69	**-0.14**	2.24	2.22	**0.02**
10-E	1.38	1.88	**-0.50**	1.95	2.41	**-0.46**
11-H	1.72	1.80	**-0.08**	2.46	2.41	**0.05**
12-E	1.33	1.78	**-0.45**	1.71	2.03	**-0.32**

[1]In 1-A 1 represents the segment number and A represent the treatment type as explained below.
A = Mill and inlay driving lanes, re-profile transverse cracks at shoulders and fog shoulders (Typical section A).
B = Re-profile full width and 50 mm single lift overlay (Typical section B).
C = Remove and replace existing ACP and GBC at the transverse crack locations, 50 mm single lift overlay, saw cut and rout and seal transverse cracks (Typical section B and Detail 4).
D = Mill and inlay driving lanes, saw cut, rout and seal selected transverse cracks (Typical section A and Detail 3).
E = Mill and inlay transverse cracks only (Typical section C).
F = Re-profile transverse cracks and 50 mm single lift overlay (Typical section D and Detail 3).
G = Mill and fill selected transverse cracks, saw cut and rout and seal these cracks, and 2-lift 70 mm ACP overlay (Typical section E and Detail 5).
H = Re-profile transverse cracks by fine grinding and double slurry seal (Alternative to Typical section C).

<u>Group 2</u> – Intermediate Pre-Construction MIRI

- Segments 1-A, 2-B, 5-D, 7-A, 8-F, and 9-G all improved the MIRI 0.6 to 1.1 mm/m due to installation of the treatments and improved the winter MIRI in the range of ~0.7 to ~1.3 mm/m.
  - Segments 1-A, and 7-A had a treatment of mill and inlay driving lanes, and re-profile and fog shoulders. The MIRI of both these segments improved about the same (0.6 to 0.8 mm/m reduction in summer MIRI and 0.6 to 0.7 mm/m reduction in winter MIRI). The field reconnaissance indicated that transverse cracks were still heaved from 4 mm to 8 mm.
  - Segment 2-B had the same rehab treatment as segment 4-B (re-profile full width plus overlay), however the reduction in MIRI for Segment 2-B was lesser than 4-B (1.1 vs. 1.6 in summer MIRI reduction and 1.3 vs. 2.0 in winter MIRI reduction). Overall ride in 2-B was good to very good.
  - Segment 5-D, where the driving lanes were milled and inlaid and selected transverse cracks were saw-cut and rout and sealed, had an improvement in the summer MIRI of 0.7 mm/m and the winter MIRI was reduced by 0.8 mm/m. The field reconnaissance indicated that the transverse cracks were very open and heaved up to 12 mm.
  - Segment 8-F had 2-lift 70 mm overlay and had a reduction in summer MIRI of 0.8 mm/m and the winter MIRI of 1.1 mm/m. The transverse cracks as observed in the field reconnaissance were very slightly heaved.

– Segment 9-G where the transverse cracks were re-profiled and a single lift overlay placed had a reduction in the summer MIRI of 0.9 mm/m and the winter MIRI of 1.0 mm/m. The transverse cracks as observed in the field reconnaissance were heaved up to 5 mm.

<u>Group 3</u> – Lowest Pre-Construction IRI

- Segments 6-E, 10-E, and 12-E where the transverse cracks were milled and inlayed and Segment 11-H where the transverse cracks were milled and slurry sealed, had slightly adverse effect in the MIRI values, i.e., the MIRI slightly increased after the installation of the treatments.

Effects of Different Treatments on Roadway Profile in Different Seasons of the Year

The analysis of roadway profile data collected prior to and after the construction provided an opportunity to study the extent and severity of heaving in the travel direction, the change in heave profile prior and after installation of the treatments, and the effect of winter freezing/frost on heave profile. A detailed analysis of the longitudinal profile data was carried out at many transverse crack locations within the different treatments segments. Based on the analysis of the longitudinal profile in the inner wheelpath of outer lane, the heave height was reduced in the range of 13 mm to 37 mm for all treatments except where the transverse cracks were locally milled and inlaid or milled and slurry sealed. The reduction in heave height from August 2008 to October 2009 was in the range of 5 mm to 25 mm. An example of these analyses is given as Figure 12 at km 7.36 where the heave height reduced about 20 mm from March 2009 to January 2010. The reduction in heave height at this location was about 5 mm from August 2008 to October 2009.

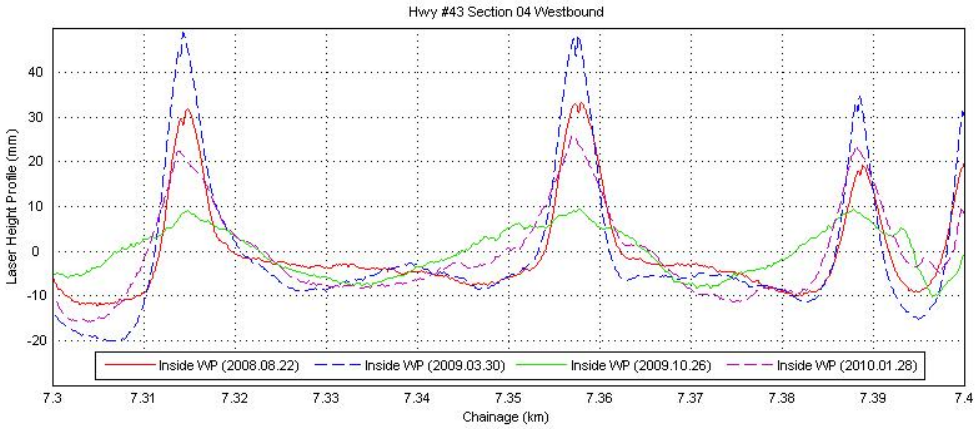


Figure 12: Hwy 43:04 WBOL IWP - change in the summer and winter longitudinal profile prior to and after construction (Segment 4-B)

The heave height of the transverse crack at km 27.94 (Segment 11-H) where the transverse cracks were milled and slurry sealed did not change. This may be the reason the MIRI for this segment did not change or nominally increased as shown in Table 2. The heave height of the transverse crack at km 28.04 (Segment 12-E) where the transverse cracks were milled and filled reduced about 2 to 3 mm from March 2009 to January 2010.

The reduction in heave height from August 2008 to October 2009 was also about 2 to 3 mm (Figure 13). Although the heave height was reduced, there were two small heaves (bumps) at each edge of the repair that may have caused the MIRI to be nominally increased.

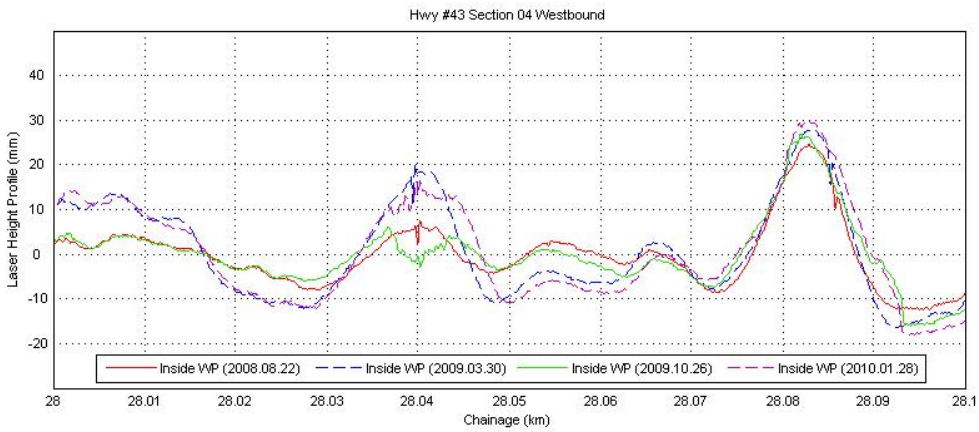


Figure 13: Hwy 43:04 WBOL IWP - change in the summer and winter longitudinal profile prior to and after construction (Segment 12-E)

The observations made are based on very short-term performance. It is recognized that some of the treatments may provide better long-term performance than others. The long-term performance will be reported in the future.

REFERENCES

[1] Karim, M., Palsat, D., Johnston, A., and Grapel, C., "Hwy 43:04 E. of Grande Prairie (WBL) Pavement Roughness Investigation", report prepared by EBA Engineering Consultants Ltd. for Alberta Transportation on February 13, 2009.

[2] Hudson, W. Ronald, "Generalized Roughness Index", Paper presented to Annual Meeting, TRB, Washington, January 1978.

[3] Transportation Association of Canada (TAC) Pavement Design and Management Guide 1997, Section 2.3.3 page 39.

[4] Alberta Transportation, "Guidelines for Assessing Pavement Preservation Treatments and Strategies", Edition 2, July 2006, available on Alberta Transportation website in pdf format.

[5] J.R. Paine & Associates Ltd., "Geotechnical Investigation/Soil Survey Highway 43 Twinning – E. of Hwy 2 to W. of Kleskun Hills", File No. GP5029-30 dated September 17, 1998.

[6] Research Council of Alberta, "Surficial Deposits of the Peace River District, Alberta, Map 28".

[7] Alberta Transportation, "Roadway Preliminary Surfacing Strategy – Final Paving, Hwy 43:04 WBL E. of Hwy 2 to W. of Bezanson", prepared by Stewart, Weir & Co. Ltd, File No. 12.026301(6), dated January 2004.

[8] Robert D. Holtz and William D. Kovacs, "An Introduction to Geotechnical Engineering", March 1981.

[9] Alberta Transportation, Peace Region, Alberta, "Design Report – Hwy 43:04 Jct. Hwy 2 to E. of Bezanson WBL km 0.000 to km 28.984", EBA No. E32101157 dated July 2009.