Long Term Performance Monitoring of the Lamont Test Road

Jim Gavin, P.Eng.
Roadway Construction Standards Specialist
Alberta Transportation

Leonard Dunn, P.Eng
Engineering Consultant

Marta Juhasz, P.Eng.
Roadway Materials Engineer
Alberta Transportation

April 2003

Paper prepared for presentation at the
Long-Life Pavements – Contributing to Canada's Infrastructure Session
of the 2003 Annual Conference of the
Transportation Association of Canada
St. John's, Newfoundland and Labrador

Abstract

Roadway agencies in northern climates are well aware of pavement deterioration problems and the accompanying maintenance expenses associated with living in a cold weather environment. One common such problem is the development of low temperature transverse cracking. To truly build long-life pavements, design procedures must be improved for the selection of better crack resistant materials.

Significant efforts were initiated in the early 1990s in support of the Canadian Strategic Highway Research Program (C-SHRP) towards the construction of three Canadian test roads, located at Lamont (Alberta), Hearst (Ontario) and Sherbrooke (Quebec). The main objective was to enhance the understanding of the asphalt cement characteristics related to low temperature cracking and to correlate the then newly published Canadian General Standards Board (CGSB) specification for asphalt cement (CAN/CGSB-16.3-M90) to field performance.

This paper deals with the design, construction, and monitoring activities of the main test road constructed in 1991 near Lamont. For that road, seven different test sections were constructed each using a markedly different grade of asphalt cement for each section. The asphalt cements were tested and classified according to the CGSB specification, as well as, the Superpave Performance Grade (PG) system. While none of the test sections included a polymer modified asphalt, two of the sections used modified asphalt cements that were oxidized or air blown for improved temperature susceptibility characteristics. On-site ambient and pavement temperature monitoring was collected during the first three winters of service. Temperature data from a local weather station has also been collected for subsequent years. The general pavement condition and transverse crack counts for the test sections have been monitored through yearly site inspections. This report also includes wheel path rutting and pavement ride quality data that has been collected by high speed survey vehicles.

After twelve winters of service, it is now possible to provide some “long term” performance results from the Lamont Test Road. The asphalt cement contribution to pavement performance is clearly evident when reviewing the wide range of observed transverse cracking frequencies. These vary from a virtual crack free pavement of two cracks per kilometre to a highly distressed pavement with 180 cracks per kilometre. Other indications on the influence of asphalt cement quality towards pavement performance are also documented.

1.0 INTRODUCTION

Highway agencies in northern climates such as Canada have long realized the economic problems associated with deteriorating asphalt pavements due to low temperature transverse cracking. Pavement researchers, many of them Canadian, have also recognized how an asphalt cement's temperature susceptibility characteristics influences pavement performance both at high (wheel path rutting) and low temperature (transverse cracking).

A brief illustration showing how the consistency or stiffness value of an asphalt cement can change based upon its temperature susceptibility characteristics is shown in Figure 1.

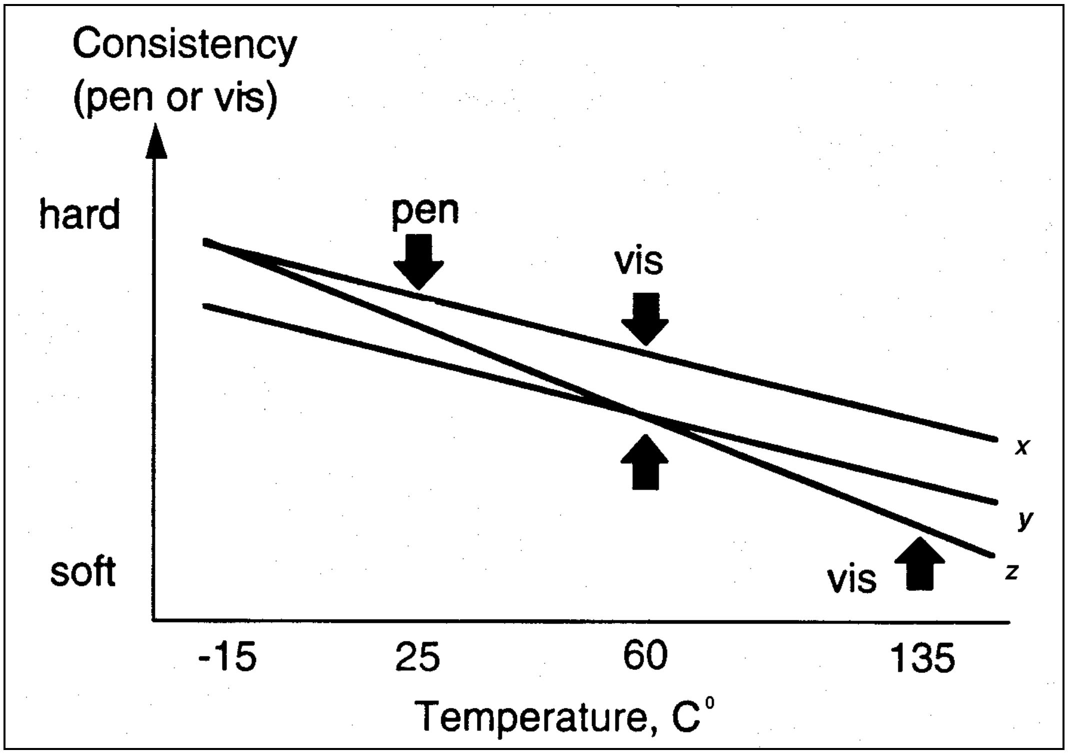

Figure 1 Temperature Susceptibility (1)

In this illustration asphalts *y* and *z* each have the same consistency values at 60°C and thus would be expected to offer similar performance, at least near this temperature. If however the consistency values for asphalt *z* are compared to asphalt *y* at both -15° and 135°C it can be concluded that asphalt *z* would not perform as well at either of these temperature extremes. In this example asphalt *y* is considered to have better temperature susceptibility characteristics than asphalt *z*.

Looking at asphalts *x* and *y,* the temperature susceptibility characteristics appear to be similar however, as asphalt *x* is a harder grade (stiffer) it would be better suited for higher trafficked pavements located in warmer environments. Likewise, asphalt *y* would offer better resistance to the formation of low temperature transverse cracks but would be less effective in resisting the formation of wheel path ruts. Together this illustrates that when selecting an asphalt cement for a particular application both, the hardness (i.e. penetration range) and temperature susceptibility characteristics need to be considered.

In early asphalt specifications, the cements were grouped together based solely upon viscosity (60°C or 135°C) or penetration (25°C) requirements. In 1990 the Canadian General Standards Board (CGSB) published the first set of national specifications, CAN/CGSB-16.3-M90 Asphalt Cement for Road Purposes, that were specifically designed to represent the quality level of asphalt cements in regards to temperature susceptibility as expressed in three groups – Group A (good performance), Group B (medium performance) and Group C (poor performance). Among other requirements the CGSB specification included consistency criteria at two temperatures; penetration @ 25°C and absolute viscosity @ 60°C. Provisions were also included to allow the use of the kinematic viscosity @ 135°C in lieu of absolute viscosity @ 60°C. It should be noted that the CGSB organization no longer supports or publishes this specification as the Committee for Road Materials, which was responsible for maintaining this specification, has ceased to exist. Despite this, agencies have continued to use the same specific wording and technical criteria contained in the original specification.[1]

1.1 Test Road Background

In the late 1980's, the Canadian Strategic Highway Research Program (C-SHRP) was initiated as a supplemental program to the Strategic Highway Research Program (SHRP) initiative undertaken in the United States. The objectives of C-SHRP were to undertake additional or complementary research, with particular emphasis on climatic conditions affecting roadway performance in Canada; to disseminate technical information resulting from the SHRP research; and to implement those SHRP products which were applicable to Canadian conditions.

A significant project within the C-SHRP program was a five-year study undertaken by EBA Engineering Consultants Ltd. (EBA) of Edmonton, Alberta entitled "Performance Correlation for Quality Paving Asphalts" (2). The primary objectives of the study were to:

- document the range of products and practices for paving asphalts in Canada,
- enhance the understanding of asphalt cement properties which influence low temperature pavement performance,
- correlate significant asphalt cement properties to low temperature pavement performance, and
- provide field correlation to support the then new CGSB specification CAN/CGSB-16.3-M90 for paving asphalts.

The final report for this study has been published by the Transportation Association of Canada (TAC) / C-SHRP (3).

As part of the EBA study, one full-scale test road and two smaller scale satellite test roads were proposed for construction in Canada. The province of Alberta volunteered to construct, instrument and monitor the performance of the full scale C-SHRP test road. The provinces of Ontario and Quebec volunteered to construct the smaller scale satellite test roads. The purpose of the satellite test roads was to extend the matrix of asphalt cements upon which the conclusions of the study could draw upon.

[1] In this report any reference to the CGSB specification implies the asphalt specifications used in 1991 and then supported by that organization.

2.0 CONSTRUCTION OF THE LAMONT TEST ROAD

2.1 General

The background and construction information on the Lamont Test Road (LTR) has been previously reported (4, 5), however some of that information is repeated here to provide a complete and up to-date summary.

The project selected by Alberta Transportation (AT) was a 12.8 km long granular base and paving project on Secondary Highway 637:02. This project is located approximately 20 km east of the town of Lamont (thereby becoming known as the Lamont Test Road) and is approximately 90 km northeast of the city of Edmonton. The surrounding terrain consists of flat to gently rolling farmland with some lightly forested sections.

Seven test sections were included and grouped towards the eastern portion of the project in an attempt to ensure uniform soil conditions with no culverts or roadway intersections. The existing embankment was constructed and lightly surfaced with gravel in 1986. The subgrade material throughout this portion consisted of a relatively uniform, low to medium plastic sandy clay till of glacial origin (CI-CL using the Unified Soil Classification System). Some CH clay was situated on the eastern limits of the projects. Due to space limitations, one of the test sections was placed in this section.

The pavement structure consisted of 100 mm of Asphalt Concrete Pavement (ACP) which was placed on top of 280 mm of a 25 mm top size crushed granular base course. The final width of roadway was 11.8 m. Each test section was between 400 m and 500 m in length and consisted solely of the appropriate test asphalt cement throughout each lane and lift of construction. Core samples were not taken within the test sections but rather in transition areas located on either end as previous experience had indicated that transverse cracks tend to occur at core locations.

2.2 Instrumentation

The Alberta Research Council (ARC) was contracted by AT to install and monitor the pavement instrumentation in order to accurately record air and pavement temperatures, as well as, the time of a first crack occurrence. In four of the test sections, crack detection loops consisting of a conductive metal foil strip were embedded between the two lifts of pavement. Each strip was placed in the west-bound lane approximately 2.2 m from the edge of pavement for a distance of 300 m. The strip was placed within a 25 mm wide by 4 mm deep groove that was routed into the first lift of pavement by a series of diamond saw blades. The four test sections chosen for instrumentation contained asphalt cements that were believed most likely to crack first.

Also placed within these four sections were thermocouples for recording ambient and ACP temperatures at depths of 12, 33, 69 and 100 mm below the pavement surface. Data loggers were used in each instrumented section to record temperatures at two-hour intervals.

2.3 Description of Asphalt Cements

Test results on the seven asphalt cements supplied to the LTR are presented in Table 1 with results for penetration at 25°C and viscosity at 60°C plotted in Figure 2. The asphalt cements used and corresponding test section numbers are as follows:

80/100 Group B, Air Blown - Test Section 1

This asphalt cement was chosen to represent a low viscosity, highly temperature susceptible crude source which had its temperature susceptibility significantly improved by air blowing. It was supplied by Imperial Oil - Esso Petroleum Canada (Esso) from their Port Moody, British Columbia refinery. The crude was obtained from the Boundary Lake, British Columbia field. The material was produced by air blowing a 400+ penetration roofing asphalt flux (RAF) to a penetration of 40 to 50 dmm and then adding sufficient RAF to bring the penetration back into the desired range of 80-100 dmm. The material initially delivered to the LTR site tested as 60 dmm and as such was below the minimum specified. Arrangements were made to have additional RAF trucked from the Port Moody refinery to the asphalt plant site where it was mixed with the existing material in order to increase the penetration to be within the specified range.

150/200 Group B - Test Section 2

This asphalt cement was chosen to represent a material of moderate to high temperature susceptibility. The asphalt cement was supplied by the Montana Refining Company (Montana) from their refinery in Great Falls, Montana. The crude consisted of a blend of approximately 40% Montana and 60% Bow River, Alberta.

300/400 Group A - Test Section 3

This asphalt cement was chosen to represent a soft asphalt cement of low temperature susceptibility. It was supplied by Esso from their Strathcona Refinery in Edmonton, Alberta. The crude was from the Cold Lake, Alberta heavy oil field. This grade is occasionally specified by AT on lower volume roadways in central and northern Alberta.

80/100 Group C - Test Section 4

This asphalt cement was chosen to represent a relatively hard (for Alberta) and highly temperature susceptible asphalt cement. It was also supplied by Esso from their Port Moody refinery. The crude for this asphalt cement was from the Redwater/Gulf fields in Alberta.

80/100 Group A, Air Blown - Test Section 5

This asphalt cement was chosen to represent a relatively hard and low temperature susceptible asphalt cement. Its temperature susceptibility was improved by air blowing during the manufacturing process. It was supplied by Husky Oil Ltd. (Husky) from their refinery in Lloydminster, Saskatchewan. The crude was obtained from the Saskatchewan gathering system of the Lloydminster heavy oil field.

150/200 Group A - Test Section 6

This asphalt cement was chosen because of its known good low temperature performance and extensive use in Western Canada. The crude source and supplier was the same as Test Section 5.

200/300 Group A - Test Section 7

This asphalt cement was the project asphalt cement selected in accordance with the then used AT design criteria based upon projected traffic loads and climatic conditions. The crude source and supplier was the same as Test Section 3.

2.4 Construction

The test sections were constructed between August 21 and September 11, 1991. The weather during this time period was favourable with daytime temperatures ranging from 15 to 25°C with no delays incurred due to inclement weather. The as-built layout of the individual test sections is shown in Figure 3.

The asphalt mix consisted of a well-graded aggregate with 100% passing the 12.5 mm sieve. The aggregate was from a river deposited gravel source and was considered to be of marginal quality due to the presence of iron nodules and sandstone. It was reasoned that the marginally quality of aggregate would not effect the low temperature evaluation of the test asphalts. Based upon the projected traffic loads and geographic location, an AT Mix Type 5 was specified using a 200-300A asphalt. The mix design followed the Marshall design method (75 blows) and called for a target asphalt content of 6.0% (by wt. of dry aggregate). This resulted in design air voids of 3.5% with 14.9% Voids Mineral Aggregate. The Marshall stability was measured to be 10,950 Newtons.

Extensive quality assurance testing was completed during construction. In general, the ACP mix met specifications with the only exception being marginally low air voids (average of 2.7%) on the field formed Marshalls. Additional details on the construction, mix design and QA testing activities have been previously reported (5).

3.0 MONITORING RESULTS

3.1 Air and Pavement Temperatures

In the Superpave design system, asphalt cements are graded according to the Performance Grade (PG) specifications. The PG designations are listed as PG *xx-yy*, where the *xx* and *yy* value represents the maximum and minimum pavement temperature requirements respectively. In the original design procedures, the minimum pavement temperature was assumed to be equal to the minimum air temperature. Work completed both in the United States under the Long Term Pavement Performance (LTPP) program (6) and in Canada under the sponsorship of TAC (7 & 8) confirmed that the pavement surface temperature is significantly warmer than the corresponding minimum air temperature. It is important to note that a substantial amount of temperature data used in the TAC study was obtained from the LTR, as well as, from the satellite test roads at Hearst and Sherbrooke. A further discussion comparing temperature results using the two procedures is included in the C-SHRP/TAC Technical Brief # 15 (8).

Ambient air and pavement temperatures were collected at the LTR for the first three winters following construction and are summarized in Table 2. Data is also reported for the nearest Environment Canada weather station, located at Vegreville, which is approximately 35 km south east of the LTR.

According to the LTPPBIND temperature database (9) the mean low temperature for the Vegreville station is –42.0°C with a standard deviation of 4.5°C. For this site the minimum expected air temperature at a 98% reliability (mean minus two standard deviations) would be –52.6°C.

Using weather data for the Vegreville station the pavement design high and low temperatures calculated using both the LTPP and TAC/Robertson algorithms would be:

Reliability (%)	Pavement Design High Temperature (°C) LTPP Equation (9)	Pavement Design Low Temperature (°C)	
		LTPP (9)	TAC (8)
50	44.2	-34.5	-31.5
70	45.9	-36.5	-33.6
90	48.4	-39.4	-36.8
98	50.9	-42.4	-40.0

Based upon this data the required Superpave Performance Grade asphalt would be a PG 52-40 (TAC low temperature equation) or 52-46 (LTPP) at a reliability value of 98%.

The lowest minimum air temperatures recorded at the LTR was during the third winter season (February 1994) with a minimum air temperature of –52.5°C. This closely matches the predicted minimum air temperature for a 98% reliability (-52.6°C). The lowest minimum pavement temperature was also recorded in February 1994 and was –36.1°C at a depth of 12 mm. Since February 1994 the yearly minimum temperatures measured at the Vegreville station have been considerably milder. The next lowest value of –43.2° C was reported in February of 1997.

3.2 Transverse Cracks

Transverse crack counts have been undertaken on a yearly basis and are summarized in Table 2. Within each section the transverse cracks were classified as being either less than 1/4 width of the pavement (not counted), greater than 1/4 width of the pavement but less than 3/4 width (counted as a half width crack) or greater than 3/4 width of the pavement (counted as a full width crack). The crack counts were summed and converted to equivalent full width transverse cracks. The total number of full width transverse cracks was then divided by the length of the test section to determine the transverse crack frequency (cracks per kilometre). For some of the test sections with high crack frequencies there are minor discrepancies in the crack counts from one year to the next. For example, in test section 2 (150-200 Group B) the reported crack frequency in 1999 was 180 cracks per kilometre while in 2000 the frequency was 178. This reflects the fact that there is some subjectivity on the part of the inspectors when determining if a crack is less than 1/4 width and not counted versus greater than 1/4 width and counted as a half crack. In actuality the crack frequency for this test section has remained the same at approximately 180 cracks per kilometre since 1998.

The first winter of 1991-92 was unusually mild with a minimum air temperature of only –30.1°C and a minimum pavement temperature of –23.3°C. No transverse cracking was noticed during this period. In subsequent years the rate of crack formation varied widely throughout the various sections. During the second winter the crack frequency in both test sections 2 (150-200 Group B) and 4 (80-100 Group C) was already over 100 cracks per kilometre. The most recent results indicate that the rate of crack development seems to have stabilized and thus are felt to be representative of the long term cracking performance for this project. Photos giving a general view of each test section are shown in Figures 4 to 10.

For test sections 3 (300-400 Group A) and 7 (200-300 Group A) explanation is needed for the two sets of crack frequencies provided in Table 2. In 1995 slab samples measuring 500 X 300 mm were saw cut from the shoulder within each of the seven test sections[2]. In 1997 a transverse crack was first noticed within test section 3 (300-400 Group A) emerging from a corner where the slab sample was removed. The crack progressed slowly across the pavement surface in subsequent winters whereas during the inspection of 2002 it was judged to be a half width crack and in 2003 a full width crack. Due to its initiation at the saw cut and its subsequent slow rate of growth, it is believed that this crack would not have occurred without the presence of the saw cut. Accordingly, the reader can make their own judgment on whether test section 7 (300-400 Group A) should be considered a crack free pavement or a cracked pavement of very low frequency.

The first three cracks in test section 7 (200-300 Group A) were observed in April, 1995. At that time, these cracks were all believed to be due to frost heave action as there was a very noticeable bump at each crack, which was not the case for cracks located elsewhere in the project. The cracks were also more jagged and torn in appearance than the straight perpendicular alignment of a typical low temperature transverse crack. Again the reader can make their own judgment on whether these cracks should be excluded. The remainder of the project outside of the seven test sections was paved using the same asphalt cement contained in test section 7. In 2003, transverse crack counts were taken of this pavement. Within the 4.26 kilometres surveyed, a total of 38 equivalent full width cracks (8.9 cracks per kilometre) were counted. Of these 38 cracks, 16 were located at sites which are considered to be crack initiators, i.e. culverts, core holes or intersecting roadways and entrances.

3.3 Assessment of CGSB and Superpave Asphalt Specifications

The asphalt cements from each test section were tested at the Central Laboratory of the Ministère des Transports du Québec (11) according to the Superpave Performance Grade system (American Association of State Highway and Transportation Officials (AASHTO) MP1 Standard Specification for Performance Graded Asphalt Binder). The PG grade for each asphalt cement and Critical Temperature as reported by Robertson (9) is listed in Table 4. A further refinement on the Superpave low temperature performance models was included in the MP1a specification approved in 2001 by AASHTO. The model in this specification uses the stiffness data from the Bending Beam Rheometer (BBR) to predict thermal stress in the binder and asphalt concrete pavement. The pavement thermal stress is compared to the binder tensile strength measured using the Direct Tension Test (DTT) in order to determine the binder's critical cracking temperature, T_{cr} (12). The calibration of the revised model was done using the asphalt cement properties and the asphalt concrete pavement properties from the LTR. The T_{cr} results reported by Bouldin et al (12) for the LTR test asphalts are also listed in Table 4 and are compared to the observed cracking frequencies.

The results in Table 4 confirms that the CGSB specifications in terms of Groups A, B and C is able to differentiate, on a relative basis, between asphalts of good or poor performance in regards to the development of low temperature transverse cracks. For a given range in

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[2] The samples were later tested for thermal coefficient of contraction at the University of Calgary as part of a TAC study on low temperature pavement performance (10). Those results are not included within this report.

penetration (ex. 150-200) the Group A asphalt with 37 cracks per kilometre is clearly a better performer than the Group B asphalt with 180 cracks per kilometre. Likewise for the penetration range of 80-100, the Group A asphalt has less cracking (119 cracks per kilometre) than the Group B asphalt (143 cracks per kilometre) and the Group C asphalt (163 cracks per kilometre). Comparing the different penetration grades within the same group clearly illustrates that the softer grades (higher penetration) offer better cracking resistance. For example, the 300-400 Group A section (0 cracks per kilometre) has less cracks than the 200-300 Group A section (4cracks per kilometre) which in turn is lower than the 150-200 Group A section (37 cracks per kilometre) and the 80-100 Group A section (119 cracks per kilometre). The same comparison breaks down when examining the two Group B sections as in this case the softer 150-200 Group B section (180 cracks per kilometre) has more cracking than the 80-100 Group B section (143 cracks per kilometre). While the relative ranking capabilities of the CGSB specification is good it does not directly measure the physical properties of importance, such as stiffness, and relate those properties to in-service temperature and loading conditions. It is also generally recognized that specifications relying on viscosity and/or penetration measurements are not able to properly characterize polymer modified asphalt cements.

The Superpave PG system on the other hand is specifically designed to measure the critical physical properties of an asphalt cement and directly relate those properties to expected field performance within the anticipated range of in-service pavement temperatures. The critical temperatures for cracking as determined using MP1 and MP1a specifications are reported in Table 4. In each case the ranking by critical temperature matches relatively well with the observed cracking frequencies. The critical temperatures for the 300-400 Group A asphalt were reported as –38.4°C for MP1 and –39.5°C for MP1a testing. This was the only section that was crack free (excluding the transverse crack that was initiated at a saw cut) after 12 years in-service. These two temperatures match very well with the pavement low temperature design requirements of approximately -40°C and the minimum measured pavement temperature of -36.1°C.

3.4 General Pavement Condition

Site inspections of the LTR have been completed on an annual basis and have included transverse crack counts and observations of general pavement deterioration. Regular maintenance activities have consisted of crack sealing operations beginning in 1994 with repeat applications approximately every three years thereafter.

Throughout most of the project the pavement surface texture is highly pitted with surface pop-outs due to the high concentration of iron nodules and sandstone in the paving aggregate. The concentration of pop-outs is generally consistent across the mat and is only slightly more noticeable within the wheel paths. For the most part, the rate in development of the surface pop-outs has stabilized with little further progression noticed within the past several years. The pitted surface is not yet considered be a significant performance problem. The exception being isolated locations where pavement segregation in combination with the detrimental aggregate has resulted in raveling and the formation of potholes which have been subsequently patched.

Between most of the test sections there is little difference in appearance in surface texture except for the oxidized asphalts used in test sections 1 and 5. In test section 1 (80-100 Group B) a minor but noticeable increase in the concentration of pop-outs and overall coarseness in surface texture can be observed. The pavement surface texture in test section 5 (80-100 Group A) is in much worse condition. General aggregate loss has occurred throughout all of the test section with isolated areas of severe raveling. In 2002 a significant number of spray patch repairs were placed solely within this test section. The spray patching was applied to both deteriorated cracks and areas of general aggregate loss. The deteriorated pavement condition observed within the two test sections using oxidized asphalts, particularly test section 5, raises questions on the merits of using such technology. It should be noted that each of the oxidized asphalt cements used on the LTR were not regular commercial products but were specifically produced for the investigative purposes of this project.

Longitudinal cracking was noticed throughout almost the entire project at the centre-line joint between mats. Within most of the test sections the crack opening at the centre-line joint was generally from 2 to 10 mm in width and contained minor braiding. In test section 1 (80-100 Group B) a slightly greater amount of braiding was noticed, while in test section 4 (80-100 Group C) the crack was slightly wider at 10 – 15 mm.

A longitudinal “centre-of-paver” crack was noticed throughout most of the project in the outer wheel path (offset of 3.5 m). The deterioration of this crack varied widely throughout the test sections and can be broadly classified as minor, moderate or severe. Minor deterioration was identified in sections 2 (150-200 Group B) and 7 (200-300 Group A). In each case the centre-of-paver crack was narrow in width (2 to 10 mm) with little further deterioration in terms of secondary cracking or braiding. The centre-of-paver crack in test sections 3 (300-400 Group A) and 7 (200-300 Group A) was judged to have more moderate deterioration. In test section 3 this crack was still only 2 to 10 mm in width however throughout the entire test section secondary cracks were observed that were immediately adjacent to (within 30 mm on each side) and parallel to the centre-of-paver crack. In test section 4 (80-100 Group C) the centre-of-paver crack also had some secondary cracking and instances of braiding along with the occasional spalling. The centre-of-paver crack in the final three sections was significantly more deteriorated. In test section 1 (80-100 Group B) the crack was significantly wider (50 mm and wider) as the surrounding pavement either raveled or spalled out. Large pools of crack sealant material were still intact within this crack. The majority of the centre-of-paver crack in test section 5 (80-100 Group A) has been repaired by spray patching. Portions of this crack that were not spray patched showed some signs of secondary cracking and braiding. Test section 6 (150-200 Group A) is located towards the eastern limit of the project. More serious structural problems in regards to fatigue cracking and wheel path rutting were observed in this section making a description of the centre-of-paver crack meaningless. This is further discussed later in this report.

Pavement roughness and wheel path rutting data for the eastbound lane of the LTR is shown in Figure 11. The data was collected in June 2002 using hi-speed survey equipment. Wheel ruts were measured using an eleven-point laser bar while the IRI roughness data was collected using a Class II inertia profiler operated at 80 km per hour. The IRI and rut data measurements were averaged over a distance of 50 metres. The data in the top graph of Figure 11 represents the average rut depth over a distance of 50 metres. The middle graph represents the maximum rut depth measured within each 50 metre interval. A number of observations can be made on this data. The average rut data from km 23 to approximately km 28 is consistent between 1 and 4 mm. Located within these limits is test section 3 (300-400 Group A) which contains the softest grade of asphalt used and accordingly would expected to be the first section to exhibit problems with wheel path rutting due to mix instability. Rutting as shown in Figure 11 was between 1 to 4 mm and was measured in the field using a 1.2 metre straightedge and calibrated wedge to be between 0 and 3 mm. In all of the test sections there was no evidence of wheel path rutting due to mix instability, i.e. no observed flushing or bleeding. It is reasoned that the rut measurements shown in Figure 11 may be slightly high due to the possibility of the laser rut bar measuring isolated instances of aggregate pop-outs as wheel path ruts. This possibility appears to be more pronounced between km 28 and 29 which corresponds well to the limits of test section 5 (80-100 Group A) including the transition sections using this asphalt. In Figure 11 the maximum rut values increase quite sharply in this section while only a small overall increase is noticed in the average rut measurements. Field measurements confirm only minor rutting (0 to 5 mm) in this section that is due more to general aggregate loss than to actual pavement densification or shoving.

Within Figure 11 a sharp increase in rut measurements (both maximum and average) is noticed at approximately km 29.5. It was confirmed during a field site visit that there was indeed a noticeable increase in wheel path rutting and distress cracking at this station, especially in the westbound lane. This rutting continues eastwards to end of the project at the junction of Hwy 855. The cause for this increased deterioration does not appear to be related to the use of a particular grade of asphalt or to any deficiencies in the asphalt mix. The rutting occurred over a relatively wide bowl suggesting a weak base or subgrade. Unlike the remainder of the project, dips up to 10 mm in depth were measured at the transverse cracks within this section, again suggesting a change in the underlying subgrade. A review of test results on borehole soil samples taken of the existing subgrade prior to construction in 1991 indicated the presence of a highly plastic clay (CH - Unified Soil Classification) within these same limits which was not reported elsewhere on the project. An analysis of the falling weight deflectometer (FWD) data confirms that the average resilient modulus of the subgrade within these limits is significantly lower at approximately 20 MPa versus 40 to 50 MPa that was measured elsewhere.

The IRI roughness data shown in Figure 11 is summarized for the various test sections in Table 3. Generally speaking the sections with the least amount of cracking have the best ride quality – test sections 3 (300-400 Group A) and 7 (200-300Group A) are both the smoothest riding and have the least number of cracks. One exception is test section 2 (150-200 Group B) which has the highest number of cracks but a ride quality as smooth as the two least cracked sections. Another exception being test section 6 (150-200 Group A). This section had the overall worst ride, however was the third lowest in cracking frequency. In this case the poor ride quality is related to the previously discussed problems of rutting and distress cracking. Finally, the poor ride quality reported in test section 5 (80-100 Group A) is consistent with the field site observations of general aggregate loss and raveling along with the accompanying maintenance repairs.

4.0 CONCLUSIONS

4.1. Temperature monitoring results from the LTR indicates that a minimum air temperature corresponding to a design reliability of approximately 98% was reached in February 1994 during the third winter following construction.

4.2. After 12 winters transverse cracking has appeared in all seven test section and ranges from a low of 2 cracks per kilometre in the 300-400 Group A section to 180 cracks per kilometre in the 150-200 Group B section. It is argued that the 300-400 Group A section should be considered as crack free as the single crack in this section initiated at a saw cut where a slab sample was taken in 1995.

4.3. The crack frequency results from the LTR confirms that the CGSB asphalt specification does a good job of ranking the various cements in regards to low-temperature pavement performance.

4.4. The crack frequency results from the two test sections using oxidized asphalts confirms that some improvement in temperature susceptibility is gained through the use of this technology. However there are also strong indications that, depending upon the degree of oxidization, these asphalts are more prone to pavement durability problems.

4.5. The Superpave PG asphalt cement specifications, both MP1 and MP1a, were judged to be effective in ranking the various asphalt cements in regards to low temperature pavement performance. The Superpave PG specifications and test procedures are also better able to measure and quantify the physical properties of an asphalt cement as they relate to in-service pavement performance.

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Test Section	1	2	3	4	5	6	7
CGSB Grade	80/100B	150/200B	300/400A	80/100C	80/100A	150/200A	200/300A
Type	Air Blown				Air Blown		
Supplier	Esso	Montana	Esso	Esso	Husky	Husky	Esso
Location	Port Moody, BC	Great Falls, Montana	Edmonton, AB	Port Moody, BC	Lloydminster, SK	Lloydminster, SK	Edmonton, AB
Crude Source	Boundary Lake	Montana/ Bow River	Cold Lake	Redwater/ Gulf	Lloydminster	Lloydminster	Cold Lake
As Supplied							
Penetration, 100g, 5 s, 25°C	100	150	333	93	88	176	241
100 g, 5 s, 10°C	22	20	58	12	21	28	45
100 g, 5 s, 5°C	13	11	36	6	14	17	25
(all in dmm)							
Viscosity, Pa·s, 60°C	96	59.8	31.3	74.9	321.3	83.8	47.1
Viscosity, mm^2/s, 135°C	277	214	163	219	530	280	195
Specific Gravity	1.009	1.035	1.031	1.012	1.034	1.028	1.034
Softening Point $T_{R\&B}$, °C	-	41.9	30.7	-	49.8	36.5	-
Solubility, %	99.95	99.82	99.95	99.93	99.96	99.98	99.97
Residue after TFOT							
% Mass Loss	0.061	0.459	0.758	0.295	0.813	0.43	0.513
Penetration, dmm 25°C	65	75	156	51	48	97	125
Viscosity, Pa·s, 60°C	267.5	164.3	94.0	196.5	1391.4	186.6	125.2
Pen. % of original Absolute	65	50	47	55	55	55	52
Visc. Ratio	2.8	2.7	3.0	2.6	4.3	2.2	2.7
Calculated Temperature Susceptibility Parameters							
Penetration Index[a]	-0.65	-2.22	-1.28	-2.45	-0.05	-1.58	-1.32
PVN_{135}	-0.79	-0.74	-0.17	-1.22	0.05	-0.09	-0.30
PVN_{60}	-0.81	-0.67	0.05	-1.19	0.27	0.01	-0.08
Superpave PG Classification[b]	58-22	52-28	46-34	58-22	64-28	52-28	52-34
Note: a – Penetration Index determined by best fit from Bitumen Test Data Chart. b - Testing done at the Central Laboratory of the Ministère des Transports du Québec. (11)							

Table 1 - Lamont Test Road – Asphalt Test Results

Winter Season	Vegreville Winter Minimum Air Temp. (°C)	Temperatures From Lamont Test Road (°C)					Transverse Crack Frequency (cracks per kilometre)						
		Air[a]	Pavement Depth[a]				1	2	3	4	5	6	7
			12 mm	33 mm	66 mm	100 mm	80/100 B	150/200 B	300/400 A	80/100 C	80/100 A	150/200 A	200/300 A
1st 1991-92	-31.0	-30.1	-23.3	-21.9	-19.6	-18.9	0	0	0	0	0	0	0
2nd 1992-93	-44.0	-46.0	-32.2	-31.1	-29.5	-29.2	24	100	0	108	2	0	0
3rd 1993-94	-48.0	-52.5	-36.1	-33.8	-32.0	-29.5	60	120	0	128	13	2	0
4th 1994-95	-34.5	No temperature data collected at Lamont Test Road past March 1994.					74	126	0	136	17	2	6 (0)[b]
5th 1995-96	-42.9						83	144	0	137	33	10	6 (0)[b]
6th 1996–97	-43.2						107	156	0	161	49	26	8 (0)[b]
7th 1997–98	-40.3						125	180	0	163	56	26	8 (2)[b]
8th 1998-99	-37.3						137	180	0	162	88	29	10 (4)[b]
9th 1999-00	-31.8						129	178	0	163	91	32	10 (4)[b]
10th 2000-01	-35.2						146	183	0	162	102	33	10 (4)[b]
11th 2001-02	-33.3						146	183	1 (0)[c]	162	101	33	10 (4)[b]
12th 2002-03	-42.0						143	180	2(0)[c]	163	119	37	10 (4)[b]

[a] Air and pavement temperatures are from test section 2 for the 1st and 2nd winter and from test section 5 for the 3rd winter.
[b] Three transverse cracks in this section are thought to be frost heave related. The bracketed values have these cracks excluded.
[c] The sole transverse crack in this section initiated from a saw cut that was used to obtain a slab sample. The bracketed value has this crack excluded.

Table 2 – Winter Minimum Temperatures and Transverse Crack Frequency

Year	Test Section						
	1	2	3	4	5	6	7
2002 (IRI)	1.38	1.06	1.00	1.11	1.59	1.68	1.06
Cracks/km	143	180	0[a]	163	119	37	4[a]

a – Excludes cracks believed to be caused by factors other than low temperature.

Table 3 - Summary of IRI Roughness Data

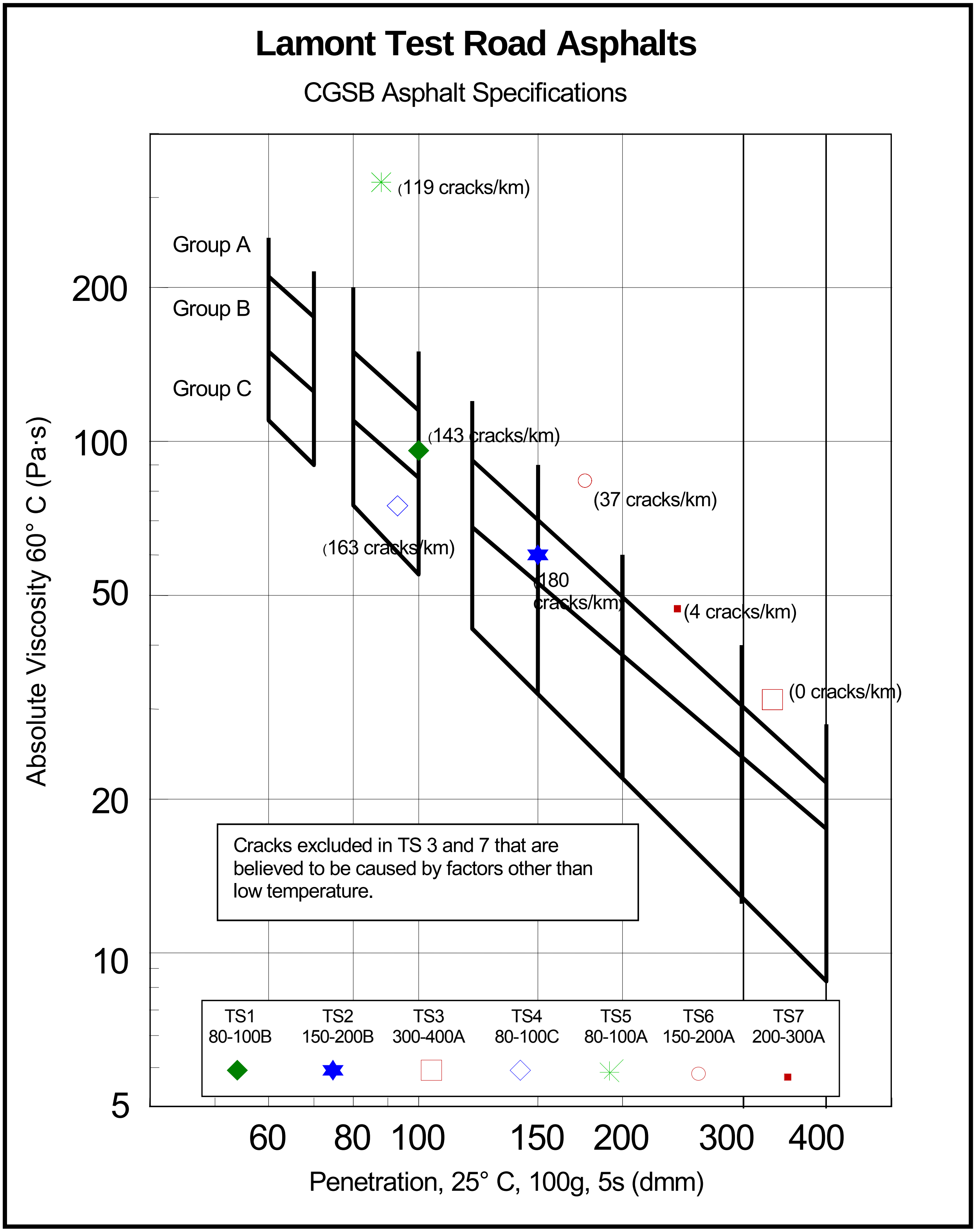
Asphalt Cement	Test Section	Superpave Performance Grade[a]	BBR Critical Temperature, (°C)[a]	T_{cr} (°C)[b]	Crack Frequency After 12 Years (Cracks per Kilometre)
300-400 Group A	3	46-34	-38.4	-39.3	0[c]
200-300 Group A	7	52-34	-35.8	-35.3	4[c]
150-200 Group A	6	52-28	-33.8	-35.1	37
80-100 Group A	5	64-28	-33.8	-31.4	119
150-200 Group B	2	52-28	-29.9	-24.1	180
80-100 Group B	1	58-22	-26.1	-34.5	143
80-100 Group C	4	58-22	-24.9	-24.6	163

a - AASHTO MP1testing done at the Central Laboratory of the Ministère des Transports du Québec as reported in reference 11. The Critical Temperature is 10ºC below the lowest temperature at which $S(60) \leq 300$ MPa and $m(60) \geq 0.300$.

b - AASHTO MP1a test results as reported in reference 12.

c – Excludes cracks believed to be caused by factors other than low temperature.

Table 4 – Critical Cracking Temperatures versus Crack Frequency



Lamont Test Road Asphalts
CGSB Asphalt Specifications
(119 cracks/km)
Group A
Group B
Group C
(143 cracks/km)
(37 cracks/km)
(163 cracks/km)
(180 cracks/km)
(4 cracks/km)
(0 cracks/km)
Cracks excluded in TS 3 and 7 that are believed to be caused by factors other than low temperature.
TS1 80-100B
TS2 150-200B
TS3 300-400A
TS4 80-100C
TS5 80-100A
TS6 150-200A
TS7 200-300A
Absolute Viscosity 60° C (Pa·s)
200
100
50
20
10
5
60
80
100
150
200
300
400
Penetration, 25° C, 100g, 5s (dmm)

Figure 2

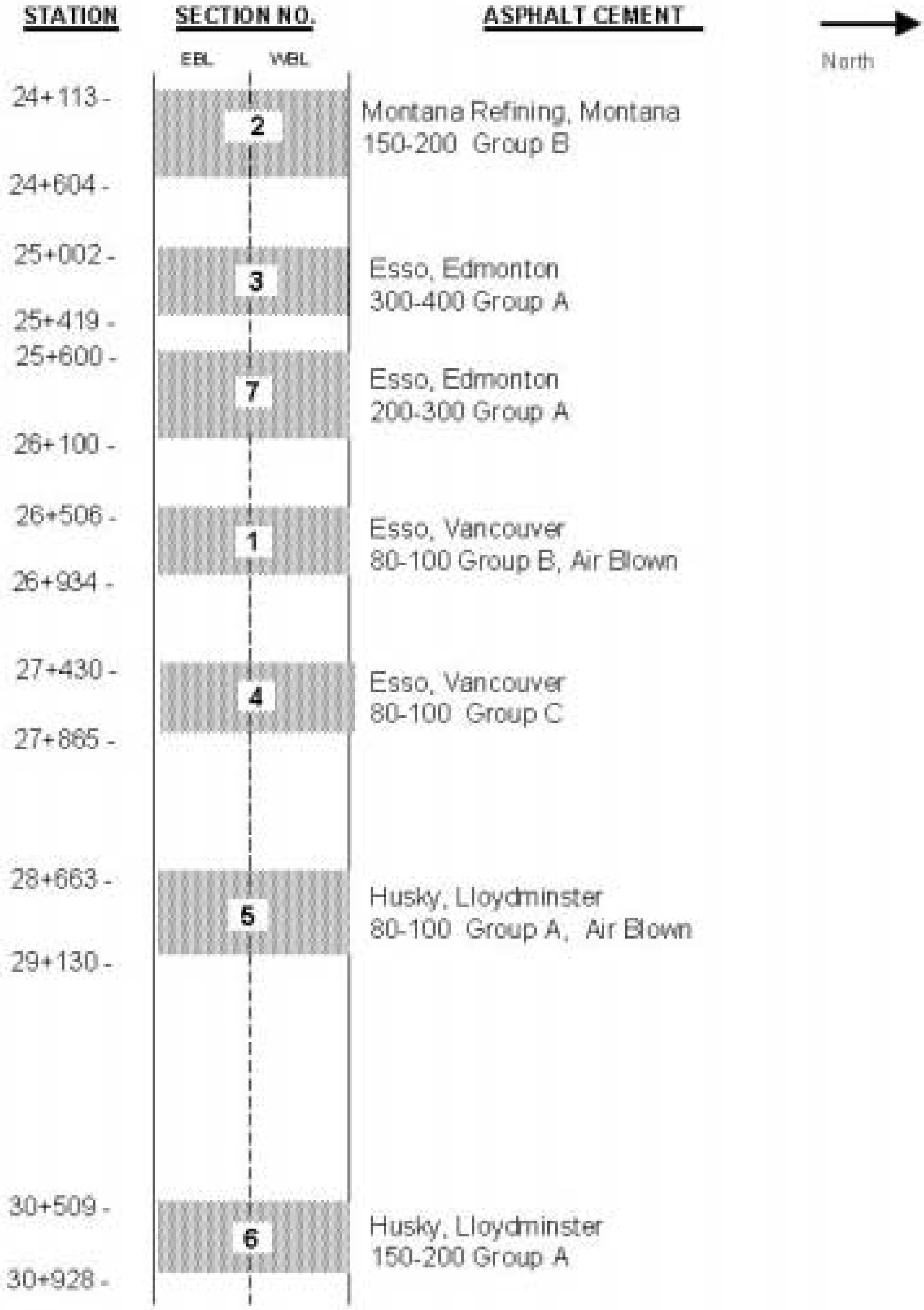


Figure 3 Lamont Test Road - Test Section Layout

Figure 4 – Test Section 1, 80-100B, 143 cracks/km

Figure 5 – Test Section 2, 150-200B, 180 cracks/km

Figure 6 – Test Section 3, 300-400A, 0 cracks/km

Figure 7 – Test Section 2, 80-100C, 163 cracks/km

Figure 8 – Test Section 5, 80-100A, 119 cracks/km

Figure 9 – Test Section 6, 150-200A, 37 cracks/km

Figure 10 – Test Section 7, 200-300A, 4 cracks/km

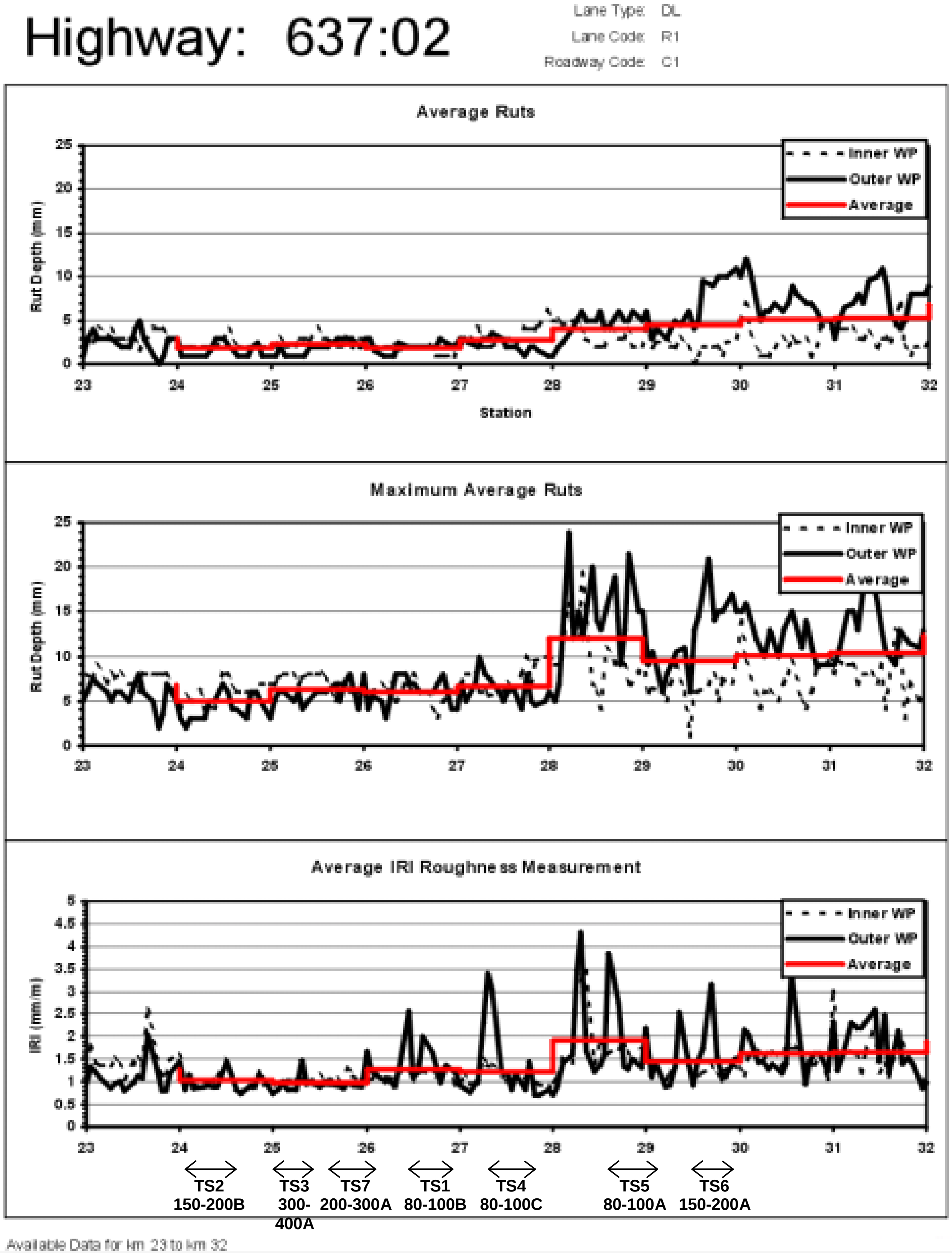


Figure 11 - 2002 IRI and Rut Data