Forensic Pavement Study: Practical Methods, Analysis and Applications

Chris Olmedo
Stantec Consulting
49 Frederick Street, Kitchener, N2H 6M7
ON, Canada
Phone: 1-519-585-7493
Fax: 1-519-579-7945
Email: Chris.Olmedo@stantec.com

Richard Korczak
Stantec Consulting
49 Frederick Street, Kitchener, N2H 6M7
ON, Canada
Phone: 1-519-585-7463
Fax: 1-519-579-7945
Email: Richard.Korczak@stantec.com

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

A forensic study was conducted in May 2008 on two test sections that are a part of the Federal Highway Administration (FHWA) Long Term Pavement Performance (LTPP) 20-year program. It was decided to investigate the causes and mechanisms of decreased pavement performance and determine what may have contributed to the differences in performance of two rural pavement sections on the Lake Ontario State Parkway in New York State.

An extensive amount of distress, deflection, environmental, construction, traffic and profile data was analyzed to establish trends as part of these investigations. Projected life cycles were calculated and compared using Empirical and Mechanistic-Empirical design methods. Issues with the pavement structure at the pavement terminal point were fully diagnosed through the use of non-destructive testing equipment coupled with validation coring and laboratory testing. The tools and methods found most useful will also be described together with examples from the forensic study. These sections failed prematurely due to a design unable to withstand environmental effects and due to lack of maintenance.

The main focus of this paper is to illustrate how routine and systematic collection of quality data combined with a forensic pavement investigation can identify changes that need to be made to materials, design and construction practices or specifications which would eliminate similar problems in the future. Optimistically, this paper will identify the methods and analysis needed to help create more efficient and cost-effective practices in the re-construction or rehabilitation of roads in these difficult economic times.

Introduction

The 20-year Federal Highway Administration (FHWA) Long Term Pavement Performance (LTPP) program is a study of pavement performance at nearly 2,500 in-service pavement sections in the United States and Canada. Sections 360801 and 360802, 152.4m (500') in length, are part of the Specific Pavement Study '*Environmental Effects in the Absence of Heavy Traffic*' SPS-8 project. The sections are located on eastbound Lake Ontario State Parkway (LOSP) approximately 2.9km West of S.R. 19 and 8 kilometers north of Hamlin, NY. The LOSP is a low volume highway with no commercial vehicles and is within sight distance of Lake Ontario. As these sections were scheduled for decommissioning, the FHWA, in conjunction with the LTTP Regional Support Contractor, Stantec Consulting and the New York State Department of Transportation (NYSDOT), decided it would be important to investigate the causes and mechanisms of decreased pavement performance and to determine what may have contributed to the differences in performance of these rural pavement sections. Both sections were constructed by the same contractor during the same time period and had the same traffic & environmental conditions.

The pavement performance indicators for these two sections show similar but different characteristics with more surface distress appearing on 360801 and 360802 having more rutting and roughness. Based on the low volumes and lack of commercial vehicular traffic, it would be expected that there would be minimal difference in the performance of these sections although there could be some benefits attributed to the thicker aggregate base and AC surface for 360802. This investigation is to examine the factors that may have contributed to the differences in performance between these two sections. Records available include construction, material sampling and laboratory analysis (done at time of construction), core samples, Falling Weight Deflectometer (FWD), Manual Distress Surveys (MDS), longitudinal and transverse profile, traffic from a continuous monitoring Weigh-in-Motion scale (WIM), environmental data from 'at site' seasonal monitoring instrumentation and a weather station installed in the area of the intersection at LOSP and SR-19. Profile, MDS, FWD, Ground Penetrating Radar (GPR), surveys were completed in May 2008 prior to selecting the locations for coring, DCP and split-spoon sampling. The locations for the surface material, DCP and split-spoon sampling, were based on a review of the FWD and MDS data. This paper documents the available historical information, forensic data collection & sampling, core sample examination, laboratory analysis & results, condition assessments, structural evaluation, findings and conclusions.

Forensic Study

As part of the forensic investigation conducted in May 2008, 100mm core samples were extracted in areas exhibiting 'no distress' and 'various levels of distress'. 150mm core samples were extracted in the mid-lane and outer wheelpath at FWD, Dynamic Cone Penetration (DCP), split-spoon and moisture sample test locations. The core samples would be used to determine the extent of damage to the asphalt surface layers, including location, width & depth of cracking, areas of visible voids, aggregate deterioration, binder adhesion or lack thereof and sufficiency of bonding between layers. At the completion of the FWD survey (conducted at 7.62-meter intervals), core locations would be selected. In the selected location, two 150mm cores (450mm apart station-wise) would be drilled to the bottom of the pavement surface, reducing the water to a trickle for the last 25mm of drilling so as not to contaminate the base material with excess moisture. The 150mm cores would be retained for measurements and laboratory testing. DCP testing was scheduled for the core hole at the FWD location with the split spoon and moisture sampling done in the nearby core hole located 450mm upstream. The 150mm cores were tested to characterize material properties and the effects of wear and aging. In addition to the Dipstick® transverse profile survey, rod and level measurements were planned to determine pavement, shoulder and grade cross-fall. Longitudinal profiles were to be collected with the ICC MDR4083 inertial profiler prior to the lane closures and sampling. Numerous photos were scheduled to document the data collection operation and site conditions. Cutting of trenches across the width of the pavement was not deemed practical for this project, based on funding limitations and the lack of commercial traffic that would result in compressions and deformations in the surface and supporting soils.

Environment

Table 1 provides the following environmental data summarized as the annual average values from the LTPP Pavement Performance Database (PPDB).

Table 1: Environmental Data

Description	Annual Average
Freezing Index (C-Days)	292
Precipitation (mm)	672
July High Air Temperature (°C)	33.5
January Low Air Temperature (°C)	-15.6
Days Above 32°C	4.9

Description	Annual Average
Days Below 0°C	105.6
Wet Days	123.2
No. of Freeze/Thaw Cycles	70.7
Annual Frost Depth (m)	0.61

The above statistics are based on 13 years of climatic data. Figures 1 to 6 provides plots summarizing the historical annual and monthly solar radiation, humidity, precipitation, and temperature. The summaries have excluded periods when the data was incomplete due to issues with the environmental instrumentation. The summary plots depict the seasonal changes that occur at the test sections located in a wet-freeze zone with a good portion of the year having wet or snowy conditions that include a number of freeze/thaw cycles with minimal frost penetration (Figure 7). The plots would also indicate that some years have more precipitation than others although the annual humidity, solar radiation and temperatures remain fairly constant.

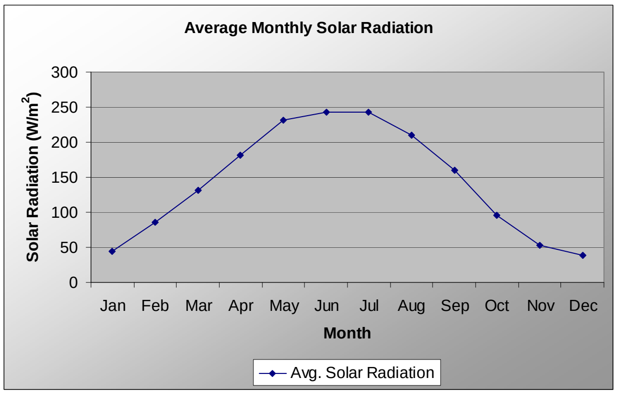

Figure 1: Average Monthly Solar Radiation

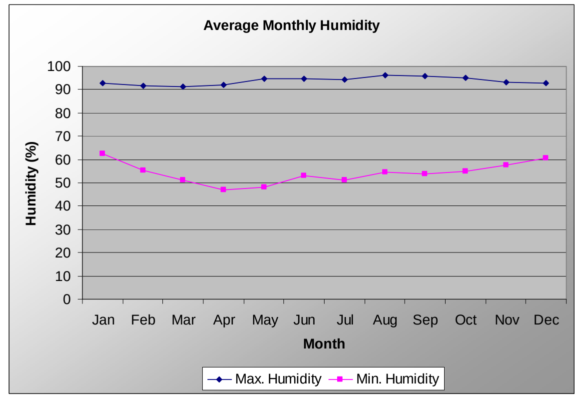

Figure 2: Average Monthly Humidity

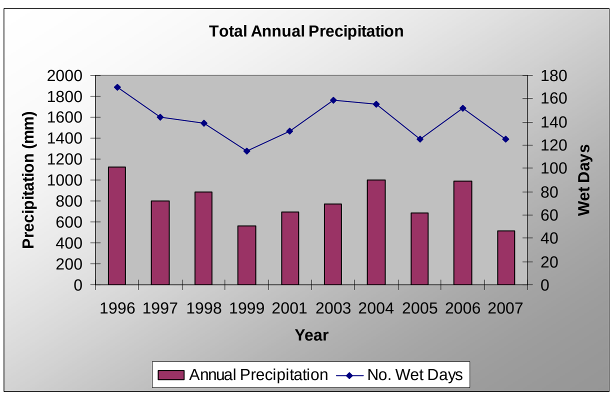

Figure 3: Total Annual Precipitation

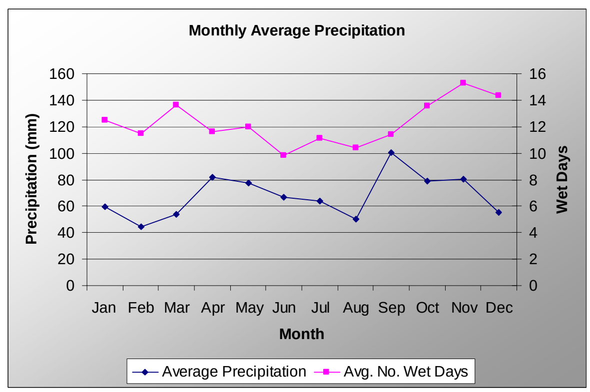

Figure 4: Average Monthly Precipitatio

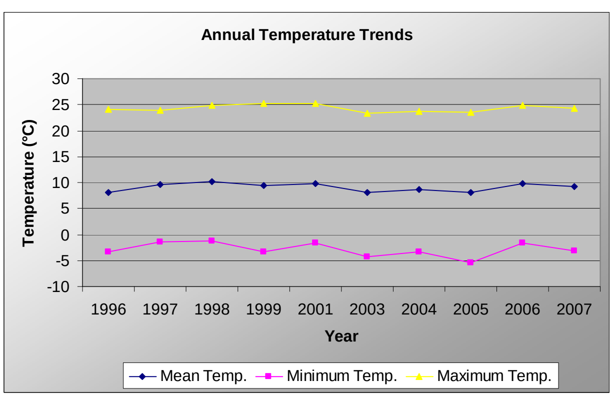

Figure 5: Annual Temperature Trends

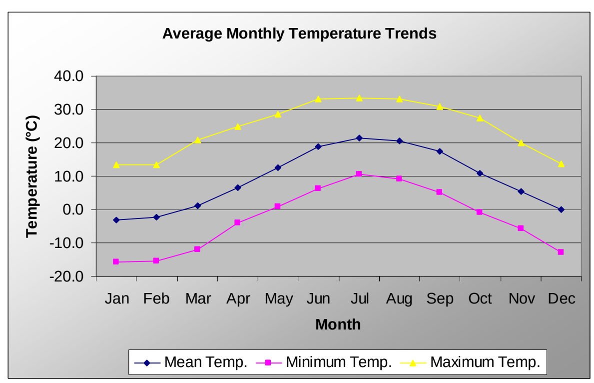

Figure 6: Average Monthly Temperature Trends

Figure 8 provides annual water table elevations from the piezometer installed at station 30.48m of section 360801. The results indicate a seasonal change in water table with the majority of samples showing a water table of less than 1m from the surface to periods when the water table fell to a depth greater than 2m from the surface. The depth to water table at the time of the forensic study was 1.08m. The median between the east and west lanes has a culvert that passes under the west bound lanes and drains towards Lake Ontario. The eastbound passing lane has a curb with catch basins draining to the median whereas the driving (slow lane) drains to the shoulder. There were no in-place drainage or permeable pavement layers included in the design or construction of the test sections.

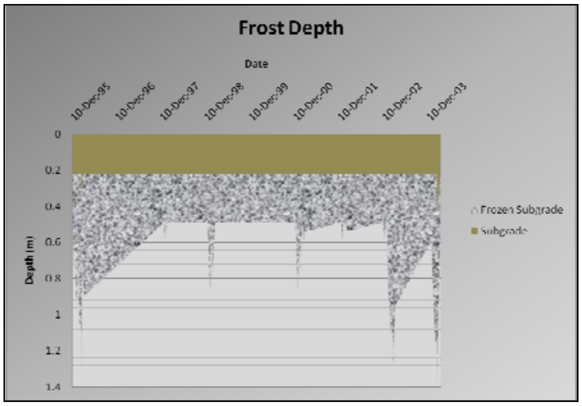

Figure 7: Frost Depth Penetration Trend (360801)

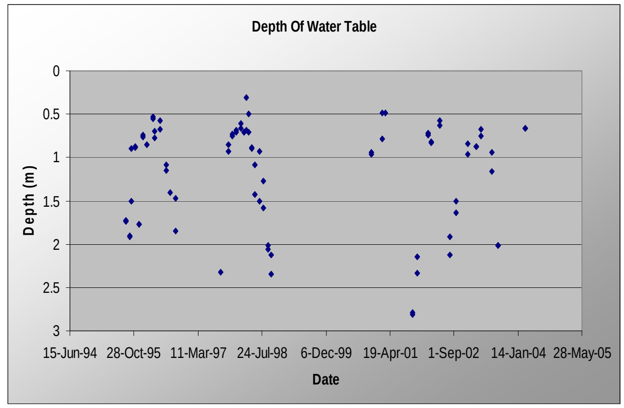

Figure 8: Annual Water Table Trend (360801)

Traffic Loading

A WIM System was installed in the eastbound driving lane 572m east of the end of the experiment section limits. Although the parkway was restricted to commercial vehicles, it was a requirement for the SPS-8 experiment to weigh and classify all individual single and tandem wheel loads. The majority of vehicles traveling this roadway would be motorcycles, cars and light trucks (Classes 1-3 of the FHWA 13-bin vehicle classification system) with the heavier vehicles being tour buses, motorhomes, towing of recreational equipment and roadway maintenance vehicles (Classes 4, 5, 6, and 8). The WIM consists of bending plates placed in the pavement so as to cover the entire 3.66m lane width. The WIM scale has been in operation since October 1995 with a numerous down periods. The repair, maintenance and calibration of this WIM have not been a high priority. Although minimal weight information has been provided, the monitoring system has provided Average Vehicle Counts (AVC) for the test sections.

The traffic information available from the LTPP database provided the following traffic information for the monitoring lane based on 13 years of estimated and 3 years of monitoring data:

- Annual Average Daily Traffic (AADT) of 1,104 vehicles/day
- Annual Average Daily Truck Volume of 10
- Annualized traffic loading 0 ESALs (Class 9)
- Annual (All Traffic) growth rate of 4.0%

Based on the traffic estimates, there were no Class 9 (18-wheel transport truck) vehicles in the SPS lane from the opening in November 1994 until the time of the forensic investigation in May 2008. However, WIM/AVC data indicates that a tiny fraction of the truck traffic were Class 9 vehicles.

Pavement Structure

The Design and as-built thicknesses are provided in Tables 2 & 3. For 360801, the as-built layer thickness is outside the specified tolerance of +/-7mm as required for this project. Some disruption of the aggregate base after final grading & tack coat, delays in delivery of asphalt and adjustments for thickness changes between the sections could have contributed to the thickness variations. For 360802, the as-built layer thicknesses were highly variable as the AC binder and aggregate base were significantly outside of the specified tolerance. Disruptions similar to 360801, in addition to changes to the asphalt supply contractor could have contributed to the thickness variations.

Table 2: Pavement Structure (360801)

Layer	Layer No.	Design Thickness (mm)	As-Built Thickness (mm)	Description
Surface Layer	3	25	30	Dense-Graded, Hot-Laid AC (Hot-Mixed, Hot-Laid Asphalt Concrete, Dense-Graded)
AC Layer Below Surface (AC Base/Binder Layer)	2	76	97	
Aggregate Base Layer	2	203	213	Crushed Gravel (Crushed Stone)
Subgrade	1	-	-	Coarse Grained Soil (Silty Sand)

Table 3: Pavement Structure (360802)

Layer	Layer No.	Design Thickness (mm)	As-Built Thickness (mm)	Description
AC Friction Coarse Surface Layer	5	25	20	Dense-Graded, Hot-Laid AC (Hot-Mixed, Hot-Laid Asphalt Concrete, Dense-Graded)
AC Layer Below Surface (Binder Course)	4	38	53	
AC Layer Below Surface (AC Binder/Base Course)	3	114	117	
Aggregate Base Layer	2	305	310	Dense-Graded Aggregate Base (Crushed Stone)
Subgrade	1	-	-	Course Grained Soil (Clayey Sand)

Construction

Reconstruction of the LOSP started on April 8, 1994. The existing pavement was removed followed by preparation and grading of the subgrade. The final grading and compaction of the subgrade was completed in July 1994. The pavement layers placed on 360802 were thicker than that of 360801; a few changes were required to accommodate the additional layer thickness. The placement and compaction of the unbound aggregate base material was completed in August 1994. For 360801, the aggregate base was placed and compacted in one lift based on a design thickness of 203mm. For 360802, the aggregate base was placed and compacted in two lifts of 203mm and 102mm based on a design thickness of 305mm. The construction traffic (trucks, paver and roller) were tracking the emulsion which lifted the aggregate which in turn resulted in disturbance and unevenness of the aggregate base prior to the placement of the asphalt base layer.

The placement of the asphalt bound layers began August 1994 with the placement of the asphalt base layer. All asphalt was sourced from a Batch Plant from Stafford, New York and transported a distance of 53km (with haul times averaging 60 minutes) to the placement location. Problems at the plant required a switch in asphalt suppliers during the placement for 360802. The second supplier, also using a drum mix plant, provided AC-15 hot mix asphalt transported a distance of 21km from Brockport, New York (with haul times averaging 30 minutes) to the placement location. The AC-15 dense graded hot mix asphalt was placed in one lift with a design thickness of 76mm and 114mm for 360801 and 360802, respectively. Problems at the plant resulted in delays in the delivery of the asphalt but the section limits were completed in as scheduled. For 360802, the placement of the binder layer with a design thickness of 38mm followed the placement of the base coarse layer using the same AC-15 asphalt mix. For both sections, an AC-20 high friction type 7F asphalt surface layer was placed in August 1994 in one lift with a design thickness of 25mm. The asphalt layers were placed at a width of 4.8m.

As part of the construction, rod & level measurements were taken at the completion of the preparation of the subgrade, aggregate base and the asphalt base and surface layers by the contractor. Nuclear densities were also taken at the completion of the compaction of the subgrade, aggregate base and asphalt surface. FWD tests were taken on the subgrade and aggregate base layers at time of construction.

The driving lanes are 3.66m wide with the outside (right) lane being monitored. The outside monitoring lane was constructed with a hot mix asphalt surface friction course over a hot mix asphalt base, with a crushed stone underlying base layer over a compacted silty sand subgrade with fragments of shale (360801) and clayey sand subgrade with fragments of shale (360802). The inside shoulder is comprised of curb with catch basins draining to a turf median. The outside lane drains to the turf shoulder; there is no subsurface drainage. The outside shoulder (adjacent to the monitored lane) is 1.52m wide with a 203mm crushed stone base and 102mm hot mix asphalt surface. The longitudinal surface joint was 3.65m from the outside shoulder lane edge joint or edge stripe. In 360802, a left turn from the left lane is located in the area of station 30.48m to 60.96m which provides access to the westbound lanes and a local roadway on the north side.

Non-Destructive Testing

As part of the forensic testing at this LTPP SPS-8 site, GPR, FWD, MDS, transverse & longitudinal profiles and elevation data were collected. This data is part of the PPDB. This data has been analyzed and the historical trends are reported as part of this document. FWD data was collected during the construction of the subgrade and aggregate base with post construction testing done on November 9, 1994. The post construction profiles were collected on September 6, 1994 and MDS on November 11, 1994. GPR data was collected on May 14, 2008 by Stantec Consulting. The following provides the results of the analysis and reports on the trends in the data from the initial data collected as part of the LTPP program to the last set of data collected as part of the forensic study.

Ground Penetrating Radar

This data was collected for the purpose of documenting the variability in thickness of the asphalt surface and aggregate base layers of the pavement structure. Figures 9 & 10 and Figures 11 & 12 provide the results of the GPR survey for the midlane and outer wheel path of sections 360801 and 360802, respectively. To determine layer thickness at the time of construction, rod and level measurements were taken at 15.2m intervals at the completion of final grade for each pavement layer. These results were used to determine the average, minimum, maximum thickness and standard deviation of each layer. In addition to the rod and level measurements, core samples taken outside the limits of the 152.4m section were also used to determine the sectional layer thickness.

The results of these surveys indicate a high variability in the thickness of the various layers with the average thickness for of the aggregate base being thicker than design specifications for 360801 and within specifications for 360802. The asphalt surface layers were found to be thicker than the design specifications in both cases. This variability was confirmed by the results from the GPR survey. Tables 4 and 5 provide a comparison of the layer thicknesses as determined from the rod and level survey and the GPR survey for each section. The results show a lower minimum and higher maximum thickness for the AC material in most cases. There is also a fairly large difference in AC thickness from centerline to edge of pavement. The midlane, on average, is thicker than the inner and outer wheelpath. The aggregate material also shows high variability as is evident by the higher standard deviation over the length and width of the sections. GPR is an excellent method of determining variability within a pavement structure with some tolerance limitations when determining actual thickness. The GPR data for this section would indicate that the construction platform was variable with the construction tolerances being outside the design specification of +/- 7mm.

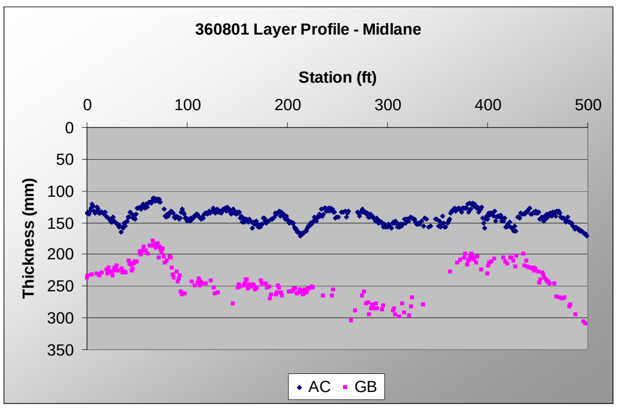

Figure 9: Midlane GPR Layer Profile (360801)

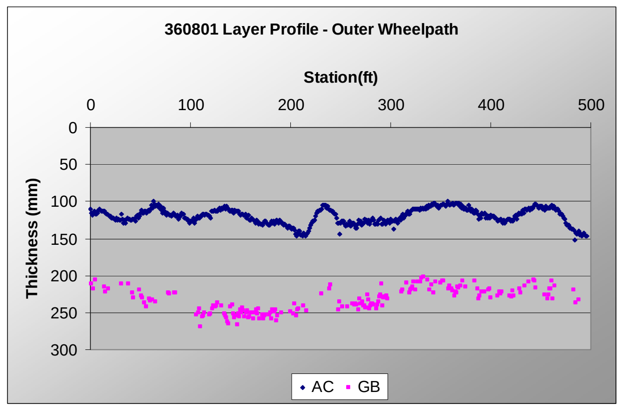

Figure 10: OWP GPR Layer Profile (360801)

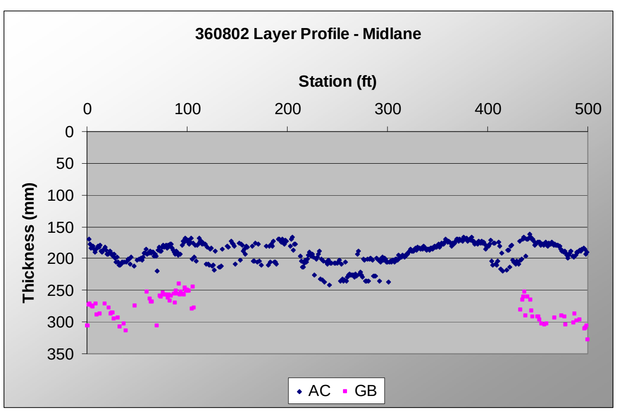

Figure 11: Midlane GPR Layer Profile (360802)

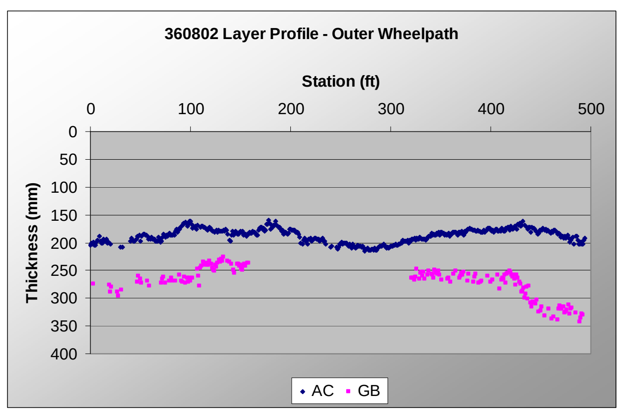

Figure 12: OWP GPR Layer Profile (360802)

Table 4: Comparison between GPR & LTPP Layer Data (360801)

Location	Layer	GPR Thickness (mm)			Standard Deviation	LTPP Layer Thickness (mm)			Standard Deviation
		Min	Max	Avg		Min	Max	Avg	
ML	AC	111.51	170.61	141.14	11.40	109.00	150.00	124.64	10.28
	Granular	179.40	308.91	238.38	29.24	213.00	250.00	224.73	10.32
OWP	AC	100.05	152.10	119.85	10.54	103.00	150.00	124.36	12.03
	Granular	201.80	269.11	233.23	16.04	219.00	265.00	236.27	14.28

Table 5: Comparison between GPR & LTPP Layer Data (360802)

Location	Layer	GPR Thickness (mm)			Standard Deviation	LTPP Layer Thickness (mm)			Standard Deviation
		Min	Max	Avg		Min	Max	Avg	
ML	AC	163.77	241.55	192.86	16.14	162.00	207.00	183.36	15.02
	Granular	239.90	327.84	277.68	21.03	287.00	338.00	311.82	14.82
OWP	AC	160.48	217.61	189.51	12.58	171.00	231.00	188.45	17.22
	Granular	225.30	343.08	271.13	28.05	293.00	329.00	311.00	13.42

Manual Distress Data Analysis Results

The historical trend for the four distress types (fatigue, longitudinal wheelpath & non wheelpath, and transverse cracking) evident on the pavement surface of both sections are provided in Figures 13 to 15. The results are from surveys conducted from 1994 to the final distress survey on May 2008. The survey results from both sections indicate distress started to appear at the centerline pavement joint in September 1997. For 360801, a small amount of longitudinal wheelpath cracking started to appear in the September 1998 survey eventually turning into fatigue cracking in July 2001. First signs of transverse cracking began to show up at this time as well. All distresses became more predominant in 2002 progressing steadily up until the final survey in 2008. Slight scraping marks on the pavement surface in the midlane and edges were first noted in the August 1995 survey and were visible throughout the life of the pavement. These marks were attributed to snowplow blade damage. For 360802, low severity longitudinal wheelpath cracking started in 1998 with the first sign of transverse cracking showing up in the 2003 survey. Fatigue or alligator cracking became predominant in 2001 at which time there was also a large increase in the length of longitudinal cracking which steadily increased until the final survey.

Longitudinal Profile Data Analysis Results

Figure 16 provides the historical IRI data for both sections. Section 360801 had an initial IRI of 1.00m/km. The historical IRI shows that the pavement roughness remained fairly constant up until 2001 and then steadily increased up to the final set of data collected in 2008 having an IRI of was 1.49m/km. The increase in roughness seems to mirror that of the accumulated distress that occurred on this section. The surface distresses on this section are mainly in the slight to moderate category with minimal distortion on a section with practically no longitudinal grade. Based on these results, the ride quality can be considered acceptable with no near term intervention required, although due to the high and increasing levels of distress, the long term preservation of this pavement section could require some remedial intervention. Section 360802 had an initial IRI of 1.07m/km. The IRI at the time of final survey was 2.26m/km having showed substantial change in roughness over time. Although there is less distress on this section when compared with 360801, it is considerably rougher. The results indicate the IRI for this section is approaching the design limit and near term corrective action should be considered.

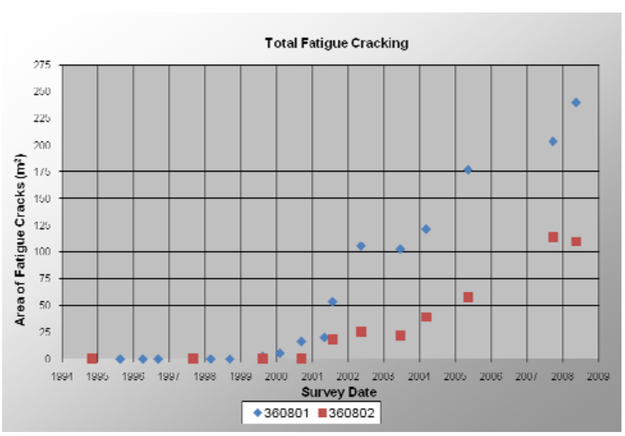

Figure 13: Historical Trend in Fatigue

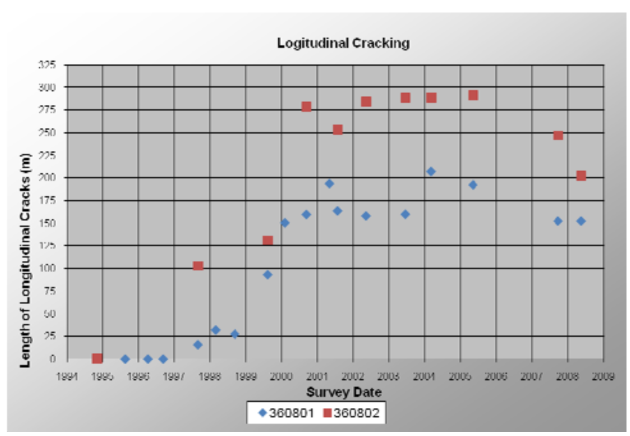

Figure 14: Historical Trend in Longitudinal Cracking

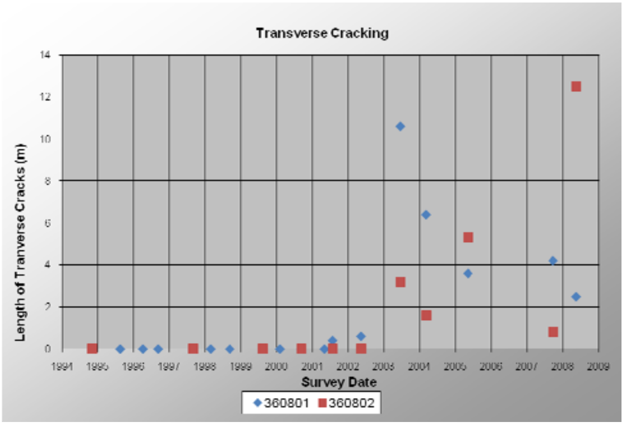

Figure 15: Historical Trend in Transverse Cracking

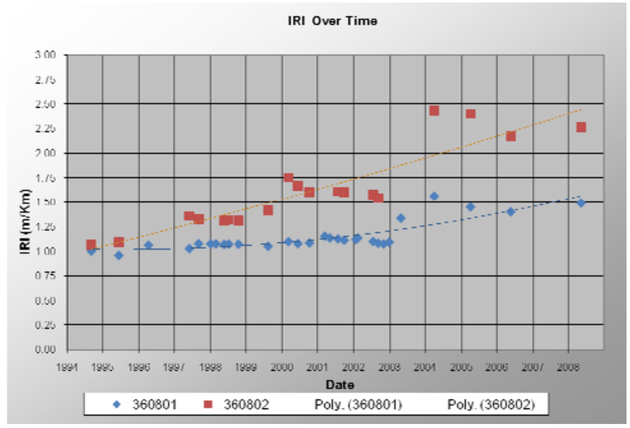

Figure 16: Historical Trend in IRI

Transverse Profile Data Analysis Results

The historical trends in rut depth from the Dipstick® transverse profiles are provided in Figure 17. At 360801, these results indicate a very slight progression in rut depth over time with the left rut in most cases being slightly deeper than the right. Figure 18 shows the rut depth data collected during the May 20 forensic study over the full 152.4m section. The average rut depth for the survey on May 20, 2008 was 3.4mm in the right wheelpath and 3.8mm in the left wheelpath. Typically the rut formations in the right wheelpath are deeper than the left as there is less lateral support, but the differences are so small in this instance that they could be considered the same. The rut depth increased since the first survey in 1995 but not to any great extent. The results of the transverse profile survey would indicate that rutting is not an issue. At 360802, rutting appeared fairly early on this section and increased steadily up until the final survey. Figure 19 shows the rut depth data collected in May 20 over the full 152.4m section. The average rut depth for the final survey was 9.4mm in the right wheelpath and 3.9mm in the left wheelpath. The results of the transverse profile survey would indicate that rutting is higher than what would be expected based on the traffic volume and lack of commercial content.

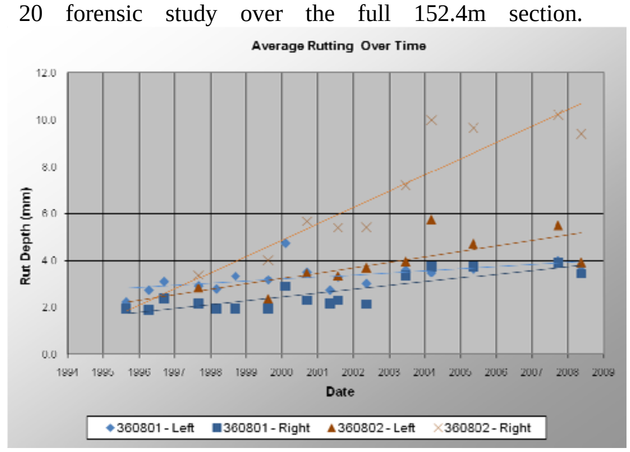

Figure 17: Graphical Presentation of Rut Depth over Time

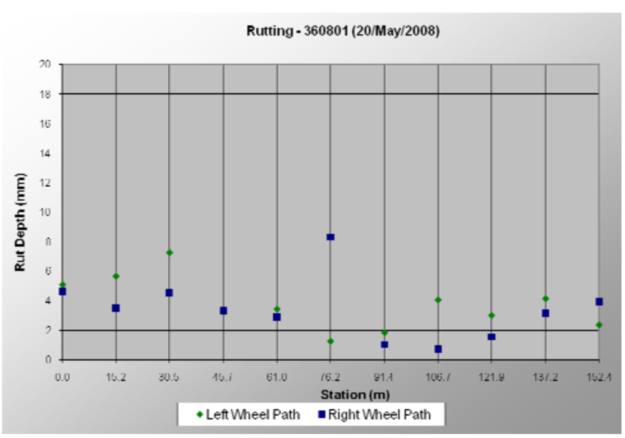

Figure 18: Rut Depth – Forensic testing (360801)

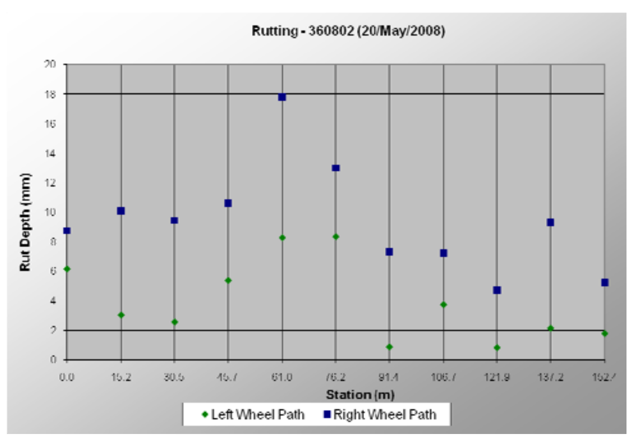

Figure 19: Rut Depth – Forensic testing (360802)

FWD Data

The FWD data was collected following the guidelines and protocols established for the LTPP program. A total of 19 drops (3 seating, 4@26kN, 4@40kN, 4@54kN and 4@72kN) were taken at each test point. The average normalized temperature corrected deflections for the 40kN equivalent loading for all the stations for both midlane and outer wheelpath were plotted with time. The surface deflection trends, as reported from the sensor located under the load plate, are provided for all stations in Figures 20 & 21 for 360801 and 360802, respectively.

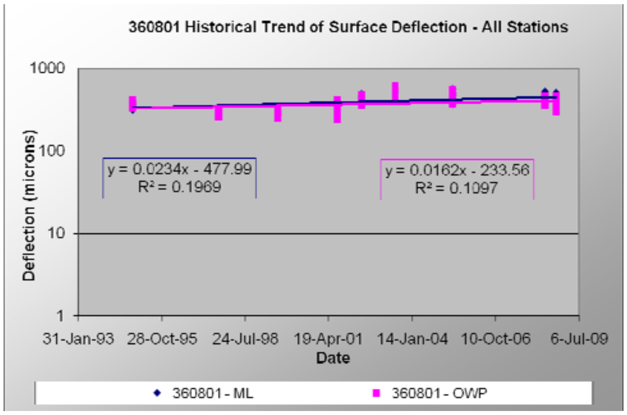

Figure 20: Historical Trend Surface Deflections (360801)

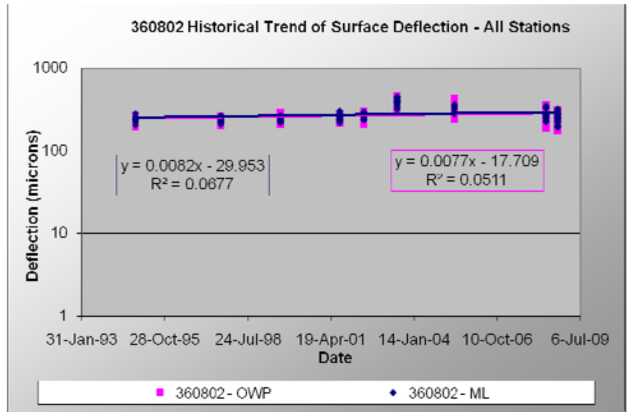

Figure 20: Historical Trend Surface Deflections (360802)

Similarly, the results representing the subgrade deflection trends, as reported from the sensor located 1.524 meters from the load plate, are provided for all stations in Figures 22 & 23 for 360801 and 360802, respectively. Section 360801 shows a continual increase in deflection indicating the pavement is losing strength as time progresses. 360802 shows a similar trend although less pronounced. The subgrade deflection trends indicate that the deflections have been very stable with time as only a slight change is evident. The results indicate only a small difference between the midlane and outer wheel path deflections, most likely due to the various distresses in each area.

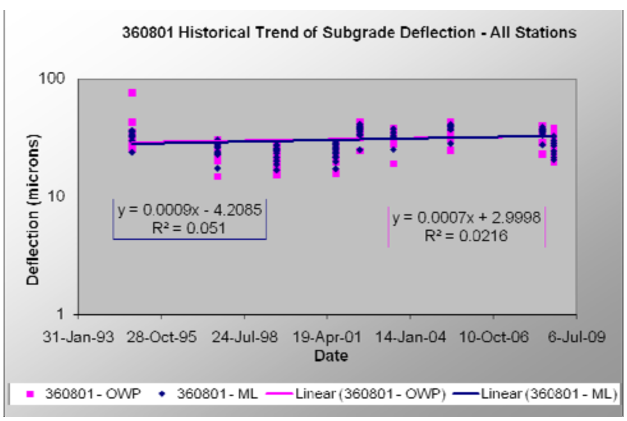

Figure 22: Historical Trend of Subgrade Deflections (360801)

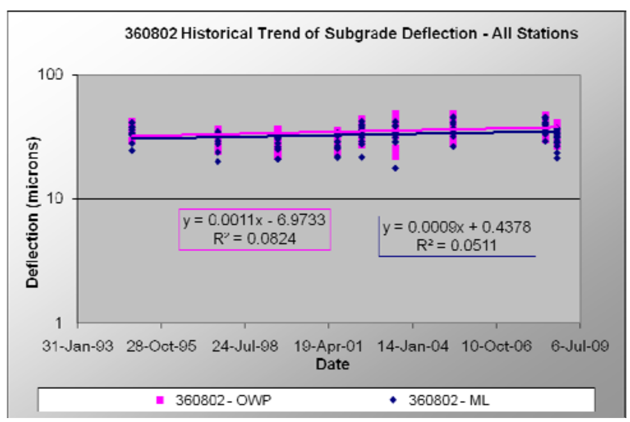

Figure 23: Historical Trend of Subgrade Deflections (360802)

The backcalculated pavement resilient moduli from the FWD deflection data is provided in Tables 6 & 7 as well as in graphical form in Figures 24 & 25 for sections 360801 and 360802 respectively. The moduli were adjusted to a reference temperature of 25°C. Figure 24 shows a slight decrease in moduli for the granular base and subgrade layers, but nothing significant. Conversely, Figure 25 illustrates the moduli of the granular base and subgrade layers decreasing in strength over time.

Table 6: Summary of Layer Moduli (360801)

Date	Lane	Layer	Modulus (MPa)
9-Nov-94	ML	AC	1355
	OWP		1531
20-May-08	ML		1105
	OWP		2783
6-Oct-08	ML		1434
	OWP		1431
9-Nov-94	ML	GB	242
	OWP		235
20-May-08	ML		137
	OWP		143
6-Oct-08	ML		174
	OWP		252
9-Nov-94	ML	SG	77
	OWP		72
20-May-08	ML		60
	OWP		59
6-Oct-08	ML		81
	OWP		68

*Note: Moduli adjusted to reference temperature of 25°C

Table 7: Summary of Layer Moduli (360802)

Date	Lane	Layer	Modulus (MPa)
9-Nov-94	OWP	AC	2098
	ML		1674
6-Oct-08	OWP		2166
	ML		2441
20-May-08	OWP		1942
	ML		1780
9-Nov-94	OWP	GB	205
	ML		212
6-Oct-08	OWP		135
	ML		121
20-May-08	OWP		162
	ML		169
9-Nov-94	OWP	SG	81
	ML		90
6-Oct-08	OWP		73
	ML		75
20-May-08	OWP		57
	ML		66

*Note: Moduli adjusted to reference temperature of 25°C

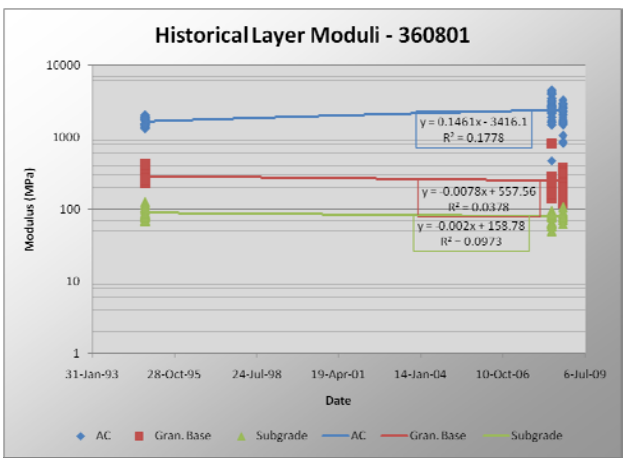

Figure 24: Historical Trend of Layer Moduli (360801)

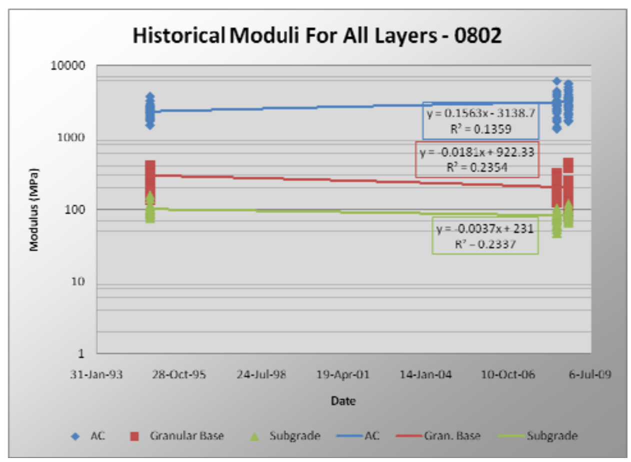

Figure 25: Historical Trend of Layer Moduli (360802)

Figures 26 and 27 show the difference in the moduli of the granular and subgrade layers for Section 360802 between the period just after construction and the time of the Forensic Study. The moduli at station 76.20m for the granular base and station 60.96m for the subgrade show a decrease in strength of between 2.5 to 3 times that of what was constructed. The strength decreases in the unbound layers can be associated to the rutting problems on Section 360802. There was minimal difference observed between the midlane and outer wheelpaths of both sections; this again is somewhat consistent with the distress observed on the surface which was located over the complete surface area rather than being primarily associated with the wheelpaths.

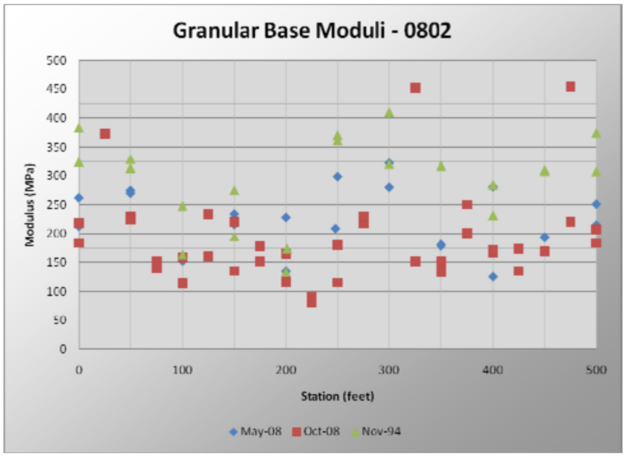

Figure 26: Granular Base Moduli (360802)

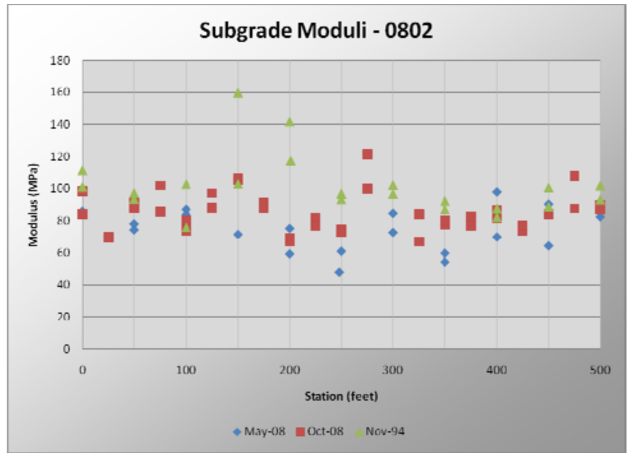

Figure 27: Subgrade Moduli (360802)

Elevation Data Analysis Results

An Eleven-Point set of levels were taken at 15.24m intervals over the 152.4m length of both sections. The results of the elevation survey are provided in Figures 28 and 29 for sections 360801 and 360802, respectively. At 360801, the results show a slight deviation in elevation at the wheelpath location with a 1.7% slope for both lanes of the pavement and a 3.6% slope from edge to just off the paved shoulder. At 360802, the results show a slight deviation in elevation at the wheelpath location with a 1.8% slope for the both lanes and a 4.5% slope from edge to just off the paved shoulder. These results would indicate sufficient slope for water runoff from the pavement surface but a slightly greater slope for the right lane could accelerate the runoff. Between the shoulder edge and just off the shoulder there is an increase in elevation for a portion of the section which could impede the runoff of the moisture from the pavement. Improvements could be considered for the abutting turf embankment area.

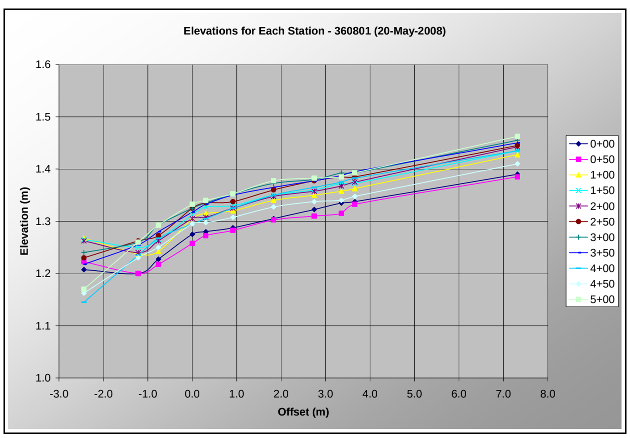

Figure 28: Results of Elevation Survey (360801)

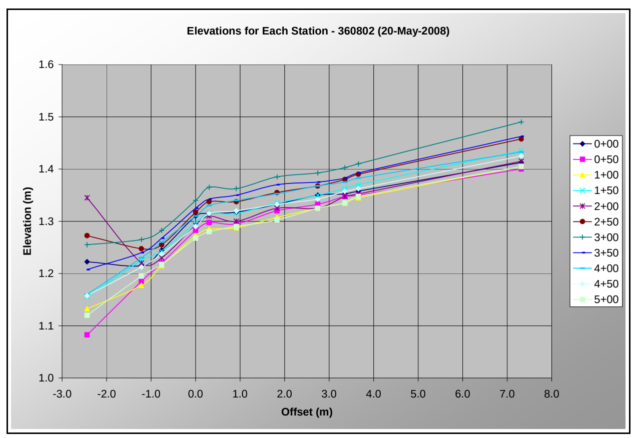

Figure 29: Results of Elevation Survey (360802)

Forensic Material Sampling and Observation

The profile, MDS and FWD surveys were completed on May 20, 2008 prior to selecting the locations for coring, DCP and split-spoon sampling. The locations for the surface material, DCP and split-spoon sampling, were based on a review of the FWD data to select representative areas of pavement response. The deflection results indicated varying pavement response over the length of the section that did not always conform to the distress and drainage observations. Three locations for sampling were selected based on variations in deflection readings, changes in drainage characteristics and localized distress. The 150mm cores that would be used for laboratory analysis and provide access for DCP and split-spoon sampling were located in the midlane and outer wheelpath at stations 30.5m, 106.7m and 152.4m. The DCP location was at the spot of the FWD test with the split spoon sampling offset by 450mm in the eastbound direction. The cores from the DCP location were selected for the laboratory analysis. The locations for the 100mm cores were based on an examination of the surface to select representative areas with cracks or no visible surface cracks that would provide core samples that could be examined to determine the extent of damage. The primary distresses for both sections were low to moderate severity alligator cracking that was in the wheelpaths, midlane, and propagating from the centerline longitudinal crack. Both sections experienced high severity centerline longitudinal cracking that had multiple cracks that progressed into each lane but were more prevalent in the SPS-8 monitored lane. In addition, Section 360801 had 5 low severity partial transverse cracks, which mainly branched off of longitudinal cracks. Section 360802 had 25 low severity partial transverse cracks. Severe cracking prior to the start of the section at station 0m and progressing into the end of the section at station 152.4m could be partially associated with the cores taken at each end of the section. In these locations the patching of the cores was deteriorated with a noticeable amount of cracking in the area of the cores.

There was a large amount of water on the shoulder area of section 360802 between stations 67.06m and 76.2m; NYSDOT personnel indicated a possible broken water pipe under the roadway at this location was under investigation. As shown above in the report, this area experienced severe rutting and backcalculation substantiated the problem with very low layer moduli for both of the granular and subgrade layers.

Cores and Core Examination

The core thickness was determined by measurements taken in 4 locations on the circumference of the core and averaged. The core condition was a visual assessment with measurement of the depth of the crack and any associated deterioration. At 360801, the cores taken in areas without any visible cracking were intact with no bonding issues between layers, and some visible voids with the binder being stiff but pliable (when poked with a knife). The asphalt surface was aged and showed signs of weathering. Raveling was present in some locations and this was especially the case where other distresses were identified. There was minor stripping, if any, at the interface of the asphalt with the aggregate base. For a number of the cores, the tack coat was bonded with the asphalt and underlying aggregate. Cores were taken in the inner wheel path where high severity distresses were full depth. The cores taken at the low severity midlane longitudinal crack were to the depth of the surface layer with only minor evidence of stripping or deterioration at the bond interface with the asphalt base material. The low severity fatigue cracking in the outer wheel path was to the depth of the surface layer but exhibited some deterioration between the interface of the surface and base. Based on the examination of the cores, roughly 70% of the cores had visible void areas primarily near the interface of the asphalt surface and base layer. The surface was substantially weathered with some raveling; only 2% of the cores had aggregate particles loose enough to be separated. Lack of bond between layers or separation due to stripping at the location of cracks was documented for 18% of the cores. All cracks identified were; top down with the low severity longitudinal cracks to the depth of the surface layer, low severity fatigue cracks penetrating approximately 20mm in the asphalt base layer and high severity longitudinal and alligator cracks being full depth. The partial transverse cracks were full depth from where it abutted the longitudinal crack, but diminished toward the end of the crack.

Cores taken at 360802 showed top-down cracking and had variable thicknesses. The crack depths ranged from 2.5mm to 66mm within the surface and binder layer with no visible distress in the AC base layer. The cores taken at a partial transverse crack branching from the centerline longitudinal crack had top-down cracking through the surface and binder layer which diminished to the surface layer at the edge of the crack. Two sets of cores were taken at the longitudinal crack in the area of the outer wheel path. The low severity longitudinal crack penetrated the surface with the moderate severity crack in both the surface and binder layers with some stripping. Based on the examination of the cores, roughly 55% of the cores had visible void areas primarily near the interface of the different AC paving layers. The surface was substantially weathered, but none of the cores had aggregate particles loose enough to be separated. There was no indication of lack of bond between layers; there was evidence of separation due to stripping at the locations of cracks, especially for the cracks that penetrated both the surface and binder layer. All cracks identified were top-down with the low severity longitudinal cracks to the depth of the surface layer and the moderate severity longitudinal cracks penetrating to the depth of the binder layer. The partial transverse crack penetrated to the depth of the AC base layer near the abutment to the longitudinal crack but diminished toward the end of the crack.

Split-Spoon Sampling & Dynamic Cone Penetrometer (DCP) Results

Table 8 provides the results of the split-spoon sampling for the three midlane and outer wheelpath locations sampled from Section 360801. The results indicate the aggregate base and subgrade materials to be on the low side. Blow counts of 25 or greater are considered to have excellent support with a blow count of 10 or less having poor support. The values from the base material can be considered rather questionable as the base was damp from the core activity, along with the core spin off causing the top 25-50mm of material to loosen.

Table 8: Summary of Split Spoon Sampling Results – 17-May-08 (360801)

Station (m)	Offset (m)	Lane	Description	Moisture Content (%)	Depth (m)		Blows/150mm					
					From	*To*	*N-count*					
30.94	1.83	ML	~200mm crushed gravel	4.0	0	0.91	9	11	9	8	9	11
			coarse-grained silty sand	13.6								
107.14	0.91	OWP	~200mm crushed gravel	4.0	0	0.91	16	15	13	7	10	42
			coarse-grained silty sand	14.0								
	1.83	ML	~250mm crushed gravel	4.0	0	0.91	12	13	11	7	6	6
			coarse-grained silty sand	12.4								
152.4	0.91	OWP	~250mm crushed gravel	5.0	0	0.91	12	10	13	10	10	11
			coarse-grained silty sand	12.5								
	1.83	ML	~200mm crushed gravel	4.0	0	0.91	12	14	11	9	10	13
			coarse-grained silty sand	11.9								

DCP tests were performed at the five locations selected from FWD tests in the midlane and outer wheelpath for the testing done in the spring and fall. The field moisture values were taken from the soil samples retrieved as part of the split-spoon sampling on May 21st with no moisture data available for October 7th. In the results, no adjustments were made to the DCP values; similarly there were no seasonal adjustment factors applied to the FWD results. The results from the DCP test indicate the aggregate base to be stiffer than the subgrade with both values seeming reasonable for the types of material and conditions at time of test. There are a number of different models available for converting the DCPI value to CBR for which different results can be obtained, therefore if this procedure is to be extensively used some local calibration is advisable.

Table 9 provides the results of the split-spoon sampling for the midlane and outer wheelpath locations sampled for section 360802. The results indicate the aggregate base and subgrade materials are poor supporting layers. The values from the base material can be considered rather questionable as the base was damp from the core activity, along with the core spin off causing the top 25-50mm of material to loosen.

Table 9: Summary of Split Spoon Sampling Results – 21-May-08 (360802)

Station (m)	Offset (m)	Lane	Description	Moisture Content (%)	Depth (m)		Blows/150mm					
					From	*To*	*N-count*					
91.74	0.91	OWP	~260mm crushed gravel	6.0	0	0.91	9	6	5	5	5	5
			coarse-grained clayey sand	16.9								
	1.83	ML	~330mm crushed gravel	5.0	0	0.91	10	7	7	5	5	7
			coarse-grained clayey sand	16.5								

DCP tests were performed at the FWD test points in the midlane and outer wheelpath. The field moisture values were taken from the soil samples retrieved as part of the split-spoon sampling. Although the field moistures were slightly above optimum there were no adjustments to the DCP results; similarly there were no seasonal adjustment factors applied to the FWD results. There was refusal for the DCP in the outer wheelpath at station 76.66m. The subgrade CBR was on average lower than that encountered for 360801

Material Properties and Laboratory Test Results

As part of the construction and testing done at the SPS-8 project in 1994, laboratory tests were conducted on the subgrade, aggregate base material, and asphalt bound layers from material samples obtained during the processing and placement of the various pavement layers. The results of the sampling and laboratory analysis that could be obtained from the LTPP database have been summarized and included in this report. As part of the forensic investigation, core samples were collected from the midlane and outer wheelpath and transported to the NYSDOT laboratory where the following tests were conducted:

- Binder extraction (% air voids, flexural creep stiffness-aged and indirect tension failure stress)
- Bulk and maximum specific gravity
- Resilient Modulus (Indirect Tension tests at 25 °C)

These tests were conducted to determine the effects of aging on the hot mix asphalt and if any of these properties were factors in the deterioration of the bound pavement layers. The material properties for the unbound layers (base and subgrade) are provided in Table 10 for Section 360801 and Table 11 for Section 360802. The subgrade was identified as silty sand for section 360801 and a sand or clayey sand (depending on location) for Section 360802. The silty sand subgrade is considered an 'active sand' as it tends to have easy infiltration of water which can result in ice lensing during the freeze periods. Both sections had the subgrade proof rolled, leveled and fine graded prior to the placement of the surface layers. The subgrade material was well compacted with the density results exceeding the requirements for both sections. The crushed stone base was placed directly on the subgrade to an average depth of 213mm for Section 360801 and 314mm for Section 360802, but both were highly variable as previously mentioned. The nuclear density tests for both sections taken at the time of construction indicate the material was not compacted within the 95% tolerance of the standard proctor test. The moisture content was below optimum which may have had an effect on the compaction; issues with water containment and drainage may have made the contractor reluctant to water down the aggregate base material during compaction. The pavement structure has shown no signs of settlement or fatigue in the bottom layers of the asphalt bound layers, which would indicate that no issues were evident with the support structure, especially with this location having a relatively high and variable water table with no external drains or drain layer in the monitoring lane.

Table 10: Material Properties – Unbound Layers (360801)

<table>
<tr><th colspan="2">Description</th><th>Granular Base @ 5+35,0.91m Offset</th><th>Subgrade @ 5+40 0.91 m Offset</th><th colspan="3">Subgrade @ 4+00 3.05 m Offset</th><th colspan="2">Subgrade @ 2+50 3.05 m Offset</th></tr>
<tr><td colspan="2">Material (Code)</td><td>Crushed Gravel (304)</td><td>Coarse-Grained Soil: Silty Sand (214)</td><td colspan="3">Coarse-Grained Soil: Silty Sand (214)</td><td colspan="2">Coarse-Grained Soil: Silty Sand (214)</td></tr>
<tr><td colspan="2">Resilient Modulus (MPa)</td><td></td><td>49.6</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td colspan="2">Lab Max. Dry Density (kg/m^3)</td><td>2419</td><td>1938</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td colspan="2">Lab Opt. Moisture Content (%)</td><td>5.0</td><td>10.0</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td colspan="2">In-situ Wet Density (kg/m^3)</td><td>2242</td><td>2197</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td colspan="2">In-situ Dry Density (kg/m^3)</td><td>2192</td><td>2108</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td colspan="2">In-situ Moisture Content (%)</td><td>2.3</td><td>4.2</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td colspan="2">Liquid Limit</td><td>16</td><td>14</td><td colspan="3">0</td><td colspan="2">23</td></tr>
<tr><td colspan="2">Plastic Limit</td><td>15</td><td>13</td><td colspan="3">0</td><td colspan="2">16</td></tr>
<tr><td colspan="2">Plasticity Index</td><td>1</td><td>1</td><td colspan="3">NP</td><td colspan="2">7</td></tr>
<tr><td colspan="2">% Gravel</td><td>70</td><td>12</td><td colspan="3">3</td><td colspan="2">20</td></tr>
<tr><td colspan="2">% Sand</td><td>22</td><td>66.1</td><td colspan="3">68.3</td><td colspan="2">48.4</td></tr>
<tr><td>% Silt</td><td>% Clay</td><td></td><td>20</td><td>8</td><td>20</td><td>8</td><td>23</td><td>8</td></tr>
<tr><td colspan="2">% Passing #200</td><td>8</td><td>21.9</td><td colspan="3">28.7</td><td colspan="2">31.6</td></tr>
<tr><td colspan="2">Max Stone Size (mm)</td><td>38.1</td><td>25.4</td><td colspan="3">12.7</td><td colspan="2">50.8</td></tr>
<tr><td colspan="2">Specific Gravity</td><td>2.831</td><td>2.72</td><td colspan="3">2.718</td><td colspan="2">2.728</td></tr>
</table>

Table 11: Material Properties – Unbound Layers (360802)

<table>
<tr><th colspan="2">Description</th><th>Granular Base @ 5+35, .91 m Offset</th><th colspan="2">Subgrade @ 5+40, .91 m Offset</th><th colspan="2">Subgrade @ 5+40, .91 m Offset</th></tr>
<tr><td colspan="2">Material (Code)</td><td>Crushed Gravel (304)</td><td colspan="2">Coarse-Grained Soils: Clayey Sand (216)</td><td colspan="2">Coarse-Grained Soils: Clayey Sand (216)</td></tr>
<tr><td colspan="2">Resilient Modulus (MPa)</td><td></td><td colspan="4">49.6</td></tr>
<tr><td colspan="2">Lab Max. Dry Density (kg/m^3)</td><td>2419</td><td colspan="4">1826</td></tr>
<tr><td colspan="2">Lab Opt. Moisture Content (%)</td><td>5</td><td colspan="4">14</td></tr>
<tr><td colspan="2">In-situ Wet Density (kg/m^3)</td><td>2265</td><td colspan="4">2078</td></tr>
<tr><td colspan="2">In-situ Dry Density (kg/m^3)</td><td>2210</td><td colspan="4">1917</td></tr>
<tr><td colspan="2">In-situ Moisture Content (%)</td><td>2.5</td><td colspan="4">8.4</td></tr>
<tr><td colspan="2">Liquid Limit</td><td>16</td><td colspan="2">12</td><td colspan="2">19</td></tr>
<tr><td colspan="2">Plastic Limit</td><td>15</td><td colspan="2">13</td><td colspan="2">14</td></tr>
<tr><td colspan="2">Plasticity Index</td><td>1</td><td colspan="2">NP</td><td colspan="2">5</td></tr>
<tr><td colspan="2">% Gravel</td><td>64.7</td><td colspan="2">2</td><td colspan="2">2</td></tr>
<tr><td colspan="2">% Sand</td><td>26</td><td colspan="2">90.2</td><td colspan="2">91.6</td></tr>
<tr><td>% Silt</td><td>% Clay</td><td></td><td></td><td></td><td>2.5</td><td>3.5</td></tr>
<tr><td colspan="2">% Passing #200</td><td>9.3</td><td colspan="2">7.8</td><td colspan="2">6.4</td></tr>
<tr><td colspan="2">Max Stone Size (mm)</td><td>38.1</td><td colspan="2">19.1</td><td colspan="2">9.5</td></tr>
<tr><td colspan="2">Specific Gravity</td><td>2.83</td><td colspan="2">2.749</td><td colspan="2">2.737</td></tr>
</table>

The tack-coat placed at the completion of the aggregate base preparation for both sections was still tacky at the time of placement of the asphalt pavement. The material properties of the aggregate used in the asphalt mix design are provided in Table 12 (360801) and in Table 13 (360802). Looking at 360801, the AC friction surface layer consists of 16% gravel with a maximum stone size of 9.5mm and 81% sand; the AC base layer had equal amounts of gravel and sand with a maximum stone size of 19mm. The core samples taken from this section indicated that the locations of cracks and associated stripping at the layer interfaces were associated with the surface layer having the higher percentage of sand and smaller maximum stone size. The materials gradations and properties for Section 360802 are the same for the AC surface and base layer of 360801 with the binder layer having the same maximum stone size as the asphalt base but with a higher stone content at 65% with 32% sand. The core samples taken from this section indicated that the locations of cracks and associated stripping at the layer interfaces were associated with the surface layer having the higher percentage of sand and smaller maximum stone size.

Table 12: Aggregate Material Properties – Bound Layers (360801)

Description	AC – Surface	AC – Base
Material (Code)	Hot Mixed, Hot Laid AC, Dense Graded (1)	Hot Mixed, Hot Laid AC, Dense Graded (1)
Layer #	4	3
% Gravel	16.0	47.0
% Sand	81.0	48.0
% Passing #200	3.0	5.0
Max Stone Size (mm)	9.5	19.1
BSG of Coarse Agg.	2.64	2.66
Absorption (%)	0.5	0.4
BSG of Fine Agg.	2.60	2.61
Absorption (%)	1.0	1.0

Table 13: Aggregate Material Properties – Bound Layers (360802)

Description	AC - Surface	AC - Binder	AC - Base
Material (Code)	Hot Mixed, Hot Laid AC, Dense Graded (1)	Hot Mixed, Hot Laid AC, Dense Graded (1)	Hot Mixed, Hot Laid AC, Dense Graded (1)
Layer #	5	4	3
% Gravel	16.0	65.0	50.0
% Sand	83.0	32.0	46.0
% Passing #200	1.0	3.0	4.0
Max Stone Size (mm)	9.5	19.1	25.4
BSG of Coarse Agg.	2.63	2.66	2.68
Absorption (%)	0.5	0.4	0.7
BSG of Fine Agg.	2.59	2.61	2.63
Absorption (%)	1.2	1.1	1.2

AC-15 binder was used for the asphalt base & binder layers with AC-20 binder used in the friction surface layer. As previously mentioned, the asphalt concrete mix using the AC-15 asphalt cement was also produced at two different batch plants for section 360802. No mention or information on the inclusion of mineral fillers or anti-stripping agents was available. Tables 14 (360801) and 15 (360802) provide a comparison of the asphalt layer properties (voids, bulk and maximum specific gravity) for the tests performed post construction and those performed as part of the forensic study. For 360801, the information available indicated the air voids post construction for the AC base layer was 6.6%. The test performed as part of the forensic study found the AC base layer to be in the range of 4.7% to 7.9% with an average of 6.2%, a very minimal change from the time of construction. The air voids for the AC surface was 8.7% at the time of construction and ranged from 8% to 12.6% with an average of 10.5% at the time of the forensic study. The high variability and increase in the air void for the AC surface is consistent with the observed weathering and raveling of this thin surface lift. When comparing the post construction air voids in the asphalt mix with those at the time of the forensic investigation for 360802, there is a slight decrease in the percentage of air void for the three AC mixes. For both sections, a comparison of the Bulk Specific Gravity (BSG) post construction and from the forensic tests shows a minimal difference between the timeframes for the AC binder and surface layers. The results are the same for the Maximum Specific Gravity (MSG) with very little change identified in the specific gravity properties.

Table 14: Comparison of Asphalt Layer Properties-Void and Specific Gravity (360801)

Sampling Date	Layer Type	Layer #	Air Voids (%)			BSG			MSG		
			Min	Max	Avg	Min	Max	Avg	Min	Max	Avg
Post-Construction ('95-'96)	AC - Base	3	6.6	6.6	6.6	2.280	2.414	2.345	2.510	2.510	2.510
	AC - Surface	4	8.7	8.7	8.7	2.156	2.241	2.206	2.416	2.416	2.416
7-Oct-08	AC - Base	3	4.7	7.9	6.2	2.278	2.366	2.329	2.466	2.506	2.483
	AC - Surface	4	8.0	12.6	10.0	2.145	2.217	2.176	2.392	2.470	2.418

Table 15: Comparison of Asphalt Layer Properties-Void and Specific Gravity (360802)

Sampling Date	Layer Type	Layer #	Air Voids (%)			BSG			MSG		
			Min	Max	Avg	Min	Max	Avg	Min	Max	Avg
Post-Construction ('95-'96)	AC - Base	3	7.3			2.250	2.471	2.360	2.510	2.563	2.545
	AC - Binder	4	8.8			2.121	2.391	2.306	2.529	2.529	2.529
	AC - Surface	5	11.1			2.135	2.169	2.154	2.422		
7-Oct-08	AC - Base	3	4.4	9.2	7.1	2.264	2.365	2.317	2.473	2.508	2.495
	AC - Binder	4	5.0	8.5	6.6	2.300	2.391	2.337	2.468	2.539	2.501
	AC - Surface	5	7.8	9.9	8.5	2.172	2.279	2.228	2.411	2.478	2.436

Design and Life Expectancy

Using the design procedure from the 2007 Mechanistic Empirical Pavement Design Guide (MEPDG), predicted levels of cracking, rutting and cumulative heavy traffic at 90% reliability were determined for 13.75 years to coincide with the Forensic study (Figure 16). Input variables for the MEPDG analysis primarily used data extracted from the LTPP database. In instances when data inputs were not available from the LTPP database, default values provided in the MEPDG program were used.

Table 16: MEPDG Results

MEPDG Design Output	360801	360802
Longitudinal Cracking	267m for 152.4-meter section (359m at Reliability)	47m for 152.4-meter section (131m at Reliability)
Alligator Cracking	93.8% bottom up (100% at Reliability)	69.4% bottom up (88% at Reliability)
AC Thermal Fracture (Transverse Cracking)	0.00m for 152.4-meter section (2.41m at Reliability)	0.01m for 152.4-meter section (2.41m at Reliability)
Rut Depth	35.4mm at Reliability (4.8mm AC, 3.1mm Base, 22.9mm Subgrade, Total 30.9mm)	27.1mm at Reliability (4.4mm AC, 2.4mm Base, 16.3mm Subgrade, Total 23.1mm)
IRI	8.7 m/km (10.1 m/km at Reliability)	3.2 m/km (4.1 m/km at Reliability)
Cumulative heavy loads	62,314	

The 20-year analysis indicated neither section would meet the reliability criteria for the full design term with the exception of thermal cracking. In particular, significant amounts of longitudinal and alligator cracking in 360801 were predicted in the early life of this thin pavement along with rapid deterioration in ride quality. For 360802, the thicker design section showed a more gradual deterioration prediction with alligator cracking to progress more readily than any of the other distresses. The predicted cumulative heavy loads, based on the default values, are higher than the monitored values, but would be typically considered when designing a rural commuter traffic roadway with low commercial content. The results from the MEPDG analysis are significantly different than those using the procedures from the AASHTO Guide for Design of Pavement Structures, 1993. Based on the material types and thickness, Table 17 compares the results.

Table 17: Empirical Results

Empirical Design Output	360801	360802
Design Structural Number (SN)	2.87	4.75
Initial Present Serviceability Rating (PSR)	3.86	3.8
1994 estimated Equivalent Single Axle Loads (ESAL's)	1,483	
Traffic growth rate	4%	
Time before this section would reach a terminal PSR of 2.5	256 years	428 years

Section Comparison

The difference between the SPS-8 sections selected is the thickness of the asphalt and aggregate base. Section 360801 is a 'thin' pavement within the SPS-8 experimental design whereas 360802 is a 'thick' pavement section. The design specification for 360801 was 102mm of AC over 203mm of aggregate base with 360802 being 178mm AC over 305mm aggregate base. Section 360801 was constructed having an AC layer thickness of 127mm comprising a 30mm AC surface friction layer and 97mm AC base layer on an aggregate base that was placed in one lift to a thickness of 213mm over silty sand. Section 360802 was constructed having an AC layer thickness of 193mm with a 23mm AC surface friction layer, 53mm AC binder layer, and 117mm AC base on an aggregate base placed in two lifts to an average thickness of 310mm over a clayey sand. The constructed thickness for both sections was different than the design thickness and was highly variable based on rod and level surveys, core sample measurements and GPR. Both sections use a conventional AC-15 and AC-20 hot mix for the asphalt base/binder and surface friction layers, respectively. The aggregate base for both sections was a crushed stone with a maximum stone size of 38mm. The sections were constructed without a pavement drainage layer or external drains relying on the slope of the pavement to drain the pavement to a turf shoulder. The AC binder and aggregate for this project followed NYSDOT specifications. Based on the information provided there were no mineral fillers and admixes included in the job mix formula.

Information from the LTPP database was used to populate the inputs and determine the predicted performance characteristics for the two pavements for both pavement methods. The MEPDG uses load spectra, climate and material characteristics to determine pavement responses and failure rates with MEPDG defaults used in instances where information was not available from the database. The predicted performance indicated that both sections indicated that a very short life span could be expected as neither would meet the 90% Reliability criteria for a 20-year design term with the exception of thermal cracking. The results from the MEPDG analysis were quite a bit different than an analysis using the procedures from the AASHTO 1993 Design Guide procedure, which had a design life expectancy greater than 100-years. The empirical design is based on structural numbers developed from material coefficients, material characteristics and traffic ESALs. In reviewing the two methods, the biggest factor in the discrepancies would have been the environmental effects that are taken into account with greater detail in the MEPDG. That being said, limited traffic inputs and slight modifications in asphalt material characteristics and their performance capabilities may have also played a factor in the MEPDG analysis. These results would indicate that engineering judgment and refinements are needed when taking into account the many variables that go into the design of a pavement. The performance of this pavement section falls somewhere in between the two analysis predictions, as there has been some structural weakening and considerable surface distress at 360801 and a fairly significant accumulation of distress, rutting in the wheelpaths and deterioration in ride quality at 360802.

The same pavement surface distresses appear on both sections but to a different magnitude and quantity. A longitudinal crack at the location of the centerline paving joint extends the length of both sections. This crack initially appeared a couple of years after construction and extended to the length of the sections in the 2000 to 2002 timeframe. The centerline longitudinal crack has multiplied to include random, alligator and partial transverse cracks that can extend to the midlane. The extension and magnitude of cracking is much greater for 360801 which has a significant amount of associated alligator cracking whereas section 360802 has a number of partial transverse cracks that initiate in the area of the centerline longitudinal crack. Alligator and longitudinal cracks appear in the wheelpath and midlane of both sections. Although there is some distinct definition of cracking in the wheelpaths, the tendency of the cracking is to be more random, which would be consistent with the low levels of traffic on these sections. The total amount of fatigue cracking recorded for section 360801 was 450m² at the time of the May 2008 survey whereas 360802 has significantly less at 110m². The pavement surface for both sections looked weathered but did not have any significant aggregate loss with 360801 showing slightly more surface deterioration. It was noted that the high points at the edge and midlane of 360801 had scrape marks from the winter maintenance plowing, which were evident after the first winter period. Both sections did not have any signs of free surface AC from bleeding or flushing.

The analysis of the historical FWD data indicated that there was minimal change in the structural capacity of the sections over time (comparing the historical trends in the overall pavement resilient moduli). The analysis also indicates that the thicker section 360802 is structurally more sufficient than 360801. A comparison of the trends in subgrade resilient moduli indicates that both

sections have a slight decline in subgrade support, with 360801 having slightly higher moduli values. Comparison of the overall pavement moduli of the two sections shows that 360802 had greater pavement strength throughout the testing period. The subgrade resilient moduli between both sections indicate reasonably similar values with 360801 having a higher rate of loss in strength. Overall, there is a fairly large scatter in the FWD data which is attributed to the variability within the section lengths and the seasonal effects of Lake Ontario including a high and variable water table.
Pavement rutting is in both wheelpaths of each section, but to a different degree of severity. For 360801, the rutting in the left wheelpath is slightly more than the right. The first survey in 1995 had a mean maximum value of 2.2mm (in the left wheelpath) which progressed to 3.8mm in the final survey in 2008. This level of rutting would indicate no major issue for this section. The rutting on section 360802 was greater with the first survey in 1997, having a mean maximum rut depth of 3.4mm (in the right wheelpath). It then progressed to 9.4mm, in the final survey in 2008. This level of rutting would indicate some possible issue with the supporting layers or asphalt material properties as there is minimal commercial traffic on this section that would result in pavement layer and/or subgrade consolidation. Strength decreases in the unbound layers from the FWD analysis appears to corroborate this issue.

The ride quality for both sections would indicate the contractors finished product was of average quality. 360801 had an initial IRI of 1.00m/km whereas 360802 had an initial IRI of 1.07m/km. The deterioration in ride quality mirrored the increase in distress on this section but also showed signs of high variability, especially in the few years, which seem to be attributed to seasonal variation. The subgrade at 360801 can be classified as an active silty sand, which under freeze/thaw conditions can experience ice lensing resulting in instability during thaw periods. The IRI at the time of the final survey in May 2008 was 1.49m/km which would be considered acceptable for the functional use of this roadway and would not require any intervention. The sandy subgrade at 360802 along with a high water table, especially since this section lacks good drainage from the monitored lane, could result in soil changes during the freeze/thaw cycle that would impact the ride quality for this section. The IRI at time of the final survey in May 2008 was 2.26m/km which would be considered approaching the terminal level for the functional use of this roadway and is approaching a level that would require corrective action.

The elevation survey indicated that both sections had pavement and shoulder slope that would be within tolerance, but the turf area that abutted the pavement shoulder in many locations was higher than the paved shoulder, impeding the drainage of water from the pavement surface.

The examination of cores taken from both sections indicated that all cracking was top-down with some stripping and deterioration evident at the interface of the surface and AC base/binder layers. The cores taken at the longitudinal centerline joint crack for 360801 were full depth whereas all the remaining cracks were partial depth. The AC base from both sections had visible voids in particular at the interface between layers but there were no lack of bonds identified. The interface of the AC bound layers with the aggregate base show minimal, if any, signs of stripping. The tack coat applied to the aggregate surface, for most of the cores examined, had bonded the surface stone to the AC base layer. The surface of 360801, which was substantially weathered, had some loose aggregate when probed with a sharp edge, whereas the surface for 360802 was firm and intact.

The analysis of the materials data did not reveal any results that would significantly affect the performance of these pavements. The post construction laboratory tests showed some difference between the binder and asphalt tests for the AC-15 mix, as the tests were done on materials sourced from two asphalt plants. All asphalt paving materials for 360801 were sourced from one plant with various portions of 360802 having asphalt supplied from a second plant. There were differences between the test results from the plants but investigation into these differences was not evaluated in this report. The mix design properties, aggregate properties, bituminous content, air voids, penetration etc. were all within the specifications acceptable to NYSDOT. The Specific Gravity test results from the forensic testing were very similar to the post construction results for the bitumen and asphalt mixes. There was minimal change in air void content for 360801 with a slight decrease identified in the air voids for the asphalt material at 360802. There was also a slight change in the stiffness properties for the surface and binder asphalt.

A review of the construction report indicated there were some issues with water containment during construction, problems with the compaction of the aggregate base layer, maintaining a uniform thickness for the aggregate base and asphalt surface layers and some delays in the delivery of asphalt due to problems at the processing plant. The delays in delivery of asphalt could have impacted section 360802 more than 360801 as materials were delivered from two different sources for 360802. For both sections, the aggregate base was highly variable with densities below 95% proctor. The asphalt surface layer thickness was also variable and outside the design specification of +/-7mm. Although not documented, there was concern that the thick single lift asphalt base layer for 360802 may not have been compacted to specification. The reporting on these problems is consistent with the findings from the core sampling, GPR and FWD data collection.

Based on the results, observations and information provided, reasons for the failures on this section could be attributed to design, lack of maintenance and environmental conditions. Although this section had curb and good drainage to the left lane and median, the turf at the right lane shoulder could have been sloped away from the pavement edge as a good portion was higher than the pavement. The slightly rutted and weathered surface has a tendency to retain water as there is minimal traffic which would help in drying out the pavement. In addition, the left lane drains through the right lane as they are both sloped in the same direction. A slight increase in pavement slope may help in this regard. The centerline joint crack may not have progressed if sealing had occurred during the initial stages. If the single crack that was observed in the fall of 2000 was sealed this may have prevented the progressions that took place thereafter. In discussion with NYSDOT staff, sealing was an inconsistent maintenance activity. Many agencies have gone away from the butt joint, using a wedge or other techniques to alleviate or reduce the construction joint cracking problem. Road salt used in winter maintenance (resulting in high soil salinity levels during the spring runoff) could have been a contributing factor in the weathering and associated low severity cracking. The cores and laboratory analysis results indicate the observed surface distresses are primarily related to failure in the AC surface layer. Based on the limited amount of traffic (with no commercial vehicles), the failures for this section would have to be associated to either poor construction and/or to environmental conditions. Although there were some issues with the construction, there were no major issues that could be associated specifically with build problems. The insufficient compaction of the aggregate base may have contributed to the rutting but no sampling or testing was done to substantiate this. For 360801, there was no indication that the AC surface was not within the material design specifications or problems with laydown or compaction. For 360802, the placement of a thick (>100mm) and variable asphalt base layer in one lift may have had some issues with compaction that would have allowed for future consolidation and rutting. This could not be determined from the results from this forensic study as no trenches were cut to examine the transverse variability. Like 360801, there was no indication that the AC surface was not within the material design specifications, although there were some differences in the test results from the materials sampled and tested from the two plants that provided asphalt to this project.

Recommendations

After 13.75 years of service, the requirement for these two sections is similar but for different reasons. Section 360801 is in need of rehabilitative action to restore the surface condition. Section 360802 is in need of maintenance/rehabilitation to correct wheelpath rutting and ride quality. A significant amount of distress could have potentially been reduced if crack sealing had been performed on the centerline construction joint crack when it progressed to the full length of the section in the 2001 timeframe. From the testing and investigations done, there was no evidence that the turf embankment, which in many locations was higher than the pavement edge, had any effect on the pavement performance. From a practice standpoint, improvement to the drainage at the edge of pavement should be considered.

A rehabilitation strategy for sections 360801 and 360802 should include milling at least 35mm, and 30mm respectively to remove the disintegrating surface to a depth that would provide a sound base to apply an overlay that would restore the structural integrity of the pavement. Repairs at the locations of the centerline joint cracks and associated transverse cracks may require some full-depth asphalt removal. Based on the information collected, both sections could benefit from geometric or drainage improvements. There does not appear to be any issue with the performance of the asphalt base, aggregate and subgrade. The traffic on these sections does not warrant a thicker AC, although this could help relieving some of the effects of the seasonal freeze/thaw for the thinner pavement section in 360801.

The use of non-destructive testing methods, such as FWD, GPR, Profilometer and MDS provide an effective means of diagnosing pavement status. When rehabilitation or major maintenance is scheduled, an opportunity exists to enhance this analysis through coring/boring and subsequent laboratory testing. Empirical and mechanistic-empirical models are available assist engineers in future materials, design and construction practices and specifications. Routine and systematic collection of quality data creates a significant resource to aid in the more accurate prediction of a pavements life cycle. This paper has hopefully identified the tools and methods found most useful to help create more efficient and cost-effective practices in the re-construction or rehabilitation of roads in these difficult economic times.