Long Term Performance and Acoustic Properties of an Aging Open Graded Friction Course Asphalt Pavement Surface

John Laxdal, AMEC Earth & Environmental
Mike Oliver, BC MoT
Mike Bishop, AMEC Earth & Environmental
Daryl Finlayson, BC MoT
Clair Wakefield, Wakefield Acoustics Ltd.

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ABSTRACT

This paper describes a long term follow up study carried out by the British Columbia Ministry of Transportation and Infrastructure (BC MoT) on two developmental paving projects constructed in the mid 1990's using Open Graded Friction Course asphalt concrete pavement (OGFC). These projects were on Vancouver Island in a wet/no freeze climate. OGFC pavements have a much coarser gradation, significantly higher air voids, and greater asphalt film thickness than conventional asphalt pavements. Benefits include improved safety due to increased skid resistance, improved surface drainage, reduced glare, and reduced spray during wet conditions. Substantial reduction in ambient traffic noise due to the noise attenuating qualities of OGFC significantly improves driver comfort and reduces driver fatigue. The reduction of noise is also a significant benefit to people living near roadway facilities which have been surfaced with OGFC.

The study focuses on two projects constructed in and near Nanaimo BC in 1995, 1996 and 1997. One was constructed in two phases on Highway 19A in the City of Nanaimo. The other project was constructed on Highway 19 which was a new highway built to service the Duke Point Ferry Terminal.

The paper presents recent and past measurement data regarding these pavement characteristics, and analysis of the long term performance of the pavement in terms of the following pavement characteristics:

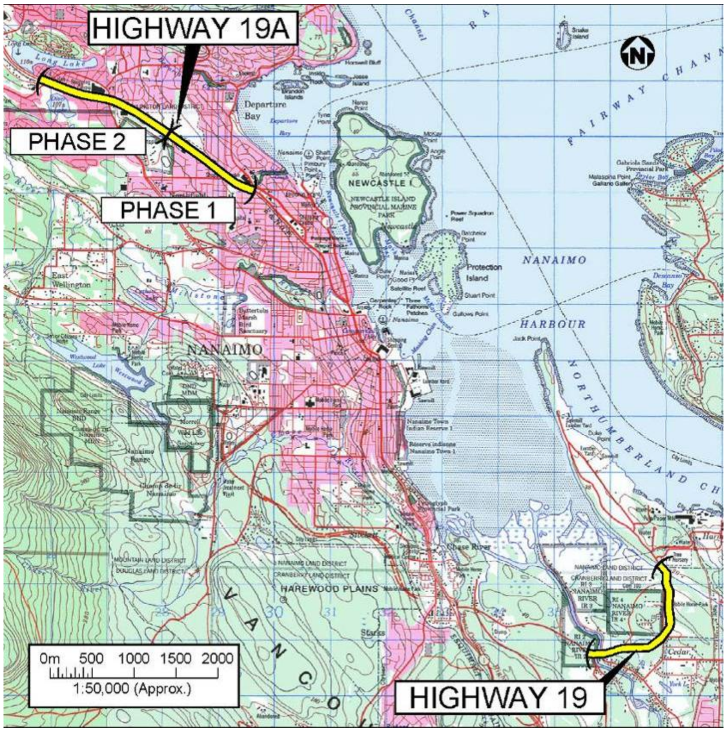
- Traffic noise attenuation
- Skid resistance
- Pavement roughness in terms of International Roughness Index
- Pavement condition in terms of Pavement Deterioration Index
- Rut depth
- Remaining pavement life as determined using AASHTO 1993 *Guide for Design of Pavement Structures* using data from Falling Weight Deflectometer testing
- Permeability

A call for traffic noise reduction is often expressed during public consultation in advance of major projects in BC. The environmental benefit of reduced noise was of significant interest to the Ministry. MoT has a noise policy dating from 1993 that identifies where project-related noise impacts warrant mitigation and that the use of "quieter pavements" can be a component of such mitigation efforts. The follow up on these past projects will be used to gauge the long term effectiveness of this surface course type and will influence policy decisions regarding asphalt surface course specifications.

1.0 BACKGROUND

In 1995 British Columbia Ministry of Transportation and Infrastructure (BC MoT) began a developmental program for Open Graded Friction Course (OGFC) asphalt concrete pavement. OGFC is differentiated from conventional dense graded asphalt concrete by its coarser aggregate gradation, interconnected air voids, and relatively thick asphalt cement film coating the aggregate particles. The interconnected air void structure results from the use of the more coarsely graded aggregate. The developmental trials included two projects: the rehabilitation of Highway 19A in north Nanaimo, British Columbia in two phases; and surfacing of a new roadway, Highway 19, which runs to Duke Point near Nanaimo. The following figure shows the location of these projects in relation to Nanaimo.

Figure 1: Test Section Locations



OGFC was of interest to BC MoT because of the following characteristics:

- increased skid resistance,
- improved surface drainage,
- sound attenuation,
- reduced glare and,
- reduced spray during wet conditions

This paper presents a 12 to 14 year follow up on the performance of these developmental projects. The present performance was assessed through the following investigations:

- Skid Resistance – British Pendulum Test (ASTM E303)
- Surface Drainage – In place permeability using apparatus meeting the requirements of *Field Permeameter No. 3* described in the National Center for Asphalt Technology (NCAT) Report 99-01, and cores taken to allow observation of the void system
- Pavement Condition – analysis of roughness, rutting and Pavement Deterioration Index data from BC MoT's Roadway Pavement Management System, and visual observation
- Pavement Strength – remaining life calculated from Falling Weight Deflectometer Data
- Sound Attenuation – measurement of *pass-by* sound pressure level

2.0 PROJECT EXTENTS AND DETAILS

Highway 19A was constructed in two phases, over two construction seasons. It was a rehabilitation of an old conventional asphalt concrete pavement. Both phases included reconstruction of failed sections of the existing pavement prior to an OGFC overlay on the full pavement, from shoulder to shoulder.

Phase 1 was placed in 1995 and consisted of rehabilitation of a four lane roadway, approximately 1.6 km in length. The existing roadway had been part of the original Vancouver Island Highway. It was in relatively poor condition with significant fatigue cracking. The fatigued areas were repaired by removal of the asphalt concrete, base repairs where required, and inlay with dense graded asphalt concrete. Approximately 20% of the existing pavement surface area was replaced. The OGFC surface layer was placed at a design application rate of 40 kg/m^2 or 18 mm thickness on this initial phase.

Phase 2 was placed in 1996 and extended from the end of Phase One to a point about 2.6 km to the north. Phase 1 had proved to have insufficient internal drainage to handle significant rainfall events and the design application rate for Phase 2 was increased to 75 kg/m^2 or +/- 33 mm thickness.

The Highway 19 segment (Duke Point Highway) was paved in 1997. This was new construction and the asphalt concrete layers consisted of 63 mm of dense graded MoT Class 1 Medium mix overlaid by 38 mm of OGFC (+/- 80 kg/m^2). The maximum aggregate size was increased slightly to compensate for the increased thickness.

The following table summarizes asphalt concrete properties from the time of construction:

Table 1: Summary of Asphalt Concrete Properties

		Highway 19 A		Highway 19
		Phase 1	Phase 2	
Gradation	**Sieve Size (mm)**	**% passing**	**% passing**	**% passing**
	19	100	100	100
	12.5	100	100	100
	9.5	95	95	88
	4.75	63	63	42
	2.36	23	23	21
	1.18	15	15	12
	0.6	9	9	7.3
	0.3	5	5	4.4
	0.15	4	4	2.7
	0.075	2.5	2.5	1.9
AC Content – % by mass of dry aggregate:		5.8	6.0	6.1
VMA (%):		+/- 25	+/- 25	28
Air Voids (%):		17	17	18
Film Thickness (µm):		18	20	20
Asphalt Cement Type:		80/100 Group A		

The traffic data for the segments was obtained from BC MoT. The following table presents this data:

Table 2: Traffic Data

Segment	Location	Average Annual Daily Traffic / Date	Truck Percentage (AASHTO Class 4 to 13)/Date
Hwy 19A	380 m north of Jingle Pot Road	34,205 / 2005	6.5 / 2008
Hwy 19	400 m south of Maughn Road	2,735 / 2007	11.6 / 2008

The climate on Vancouver Island is considered to be in a Wet No Freeze zone from a pavement engineering perspective. Annual precipitation averages 1.15 m. Roadway maintenance crews use salt but generally no abrasives for winter de-icing.

3.0 VISUAL CONDITION ASSESSMENT

The pavement surface condition was visually observed on the 27 and 28th of October 2008. The observations regarding the present pavement condition are presented in the following sections.

3.1 Highway 19A North Nanaimo Phase 1

Based on field observations, this pavement appears to have performed well and was still functioning adequately considering that it was close to the originally estimated design fatigue life. The most significant problem appeared to be structural fatigue cracking and this was generally confined to the outside lanes. Very little excavation was done in the inner lanes during the 1995 rehabilitation. The majority of the base repair work that was done in 1995 was in the outside northbound lane which carried a great deal of slow moving heavy traffic from the ferries and exhibited more distress at that time. This highway was originally two lanes wide but was widened to four lanes in the early 1970's and the centre lanes were overlaid as part of that reconstruction. The centre lanes thus have a greater asphalt concrete thickness which, combined with the higher relative truck traffic in the outside lanes, has led to better overall performance. The southbound lanes were in better condition prior to the 1995 overlay; likely because the traffic moved faster and was more regularly spaced.

A comparison of the shoulders with no traffic and the driving lanes showed only a modest amount of ravelling (photographs 1 and 2). There was some structural distress evident in the driving lanes and a minor amount of intermittent longitudinal joint failure (photograph 3). These joints were not painted with emulsion during construction in order not to impede lateral drainage. The amount of failure was minor and certainly did not suggest a need to do anything differently.

Photo 1: Driving Lanes Phase 1 - 200 m from South End – Note Structural Cracking

Photo 2: Shoulder - Phase 1 - 200 m from South End

Photo 3: Longitudinal Joint Failure - Same Area as Photo 1 and 2

Wheelpath wear was also quite evident throughout Phase 1. It was minor and not severe enough to affect the functionality of the pavement. If this was a dense graded pavement this much wear might have created a water channel and been a safety concern.

Photo 4: Wheelpath Wear Phase 1

Photo 5: Alligator Fatigue Cracking about 400 m North of Project Start

The southbound slow lanes in Phase 1 exhibited much more structural distress than any other part of this project. Very little of this lane was excavated in 1995 and this wheelpath failure is to be expected considering the advanced age of the original structure (30 years +).

3.2 Highway 19A North Nanaimo - Phase 2

Phase 2 of Highway 19A appeared to be in significantly better condition than Phase 1. This can likely be attributed to the additional thickness of OGFC that was placed as well as to some minor mix design improvements that were made as a result of lessons learned on Phase 1. There were some intermittent structural failures in areas that were not excavated and repaired prior to placement of the OGFC layer.

Photo 6: Alligator Failure North Bound Slow Lane

The next two photographs indicate the contrast between the shoulder condition and the wheelpath condition. It was evident that there was some minor ravelling in the wheelpaths.

Photo 7: Wheelpath Condition

Photo 8: Shoulder Condition

The only place where severe traffic damage was evident was in the Norwell/Jinglepot intersection with Highway 19A which is subject to many turning movements. The OGFC had been ground down to the underlying layer in a small area in the middle of the intersection. There were also a few minor areas of localized damage from wheel rims etc, and in one case where a truck box had come off a tandem truck.

3.3 Highway 19 - Duke Point Highway

The Duke Point Highway showed very little distress. What did stand out was the contrast between the wheelpaths and the rest of the pavement. The following photo illustrates this.

Photo 9: Wheelpath Contrast

At first glance this appeared to be rutting in the wheelpaths and that the winter maintenance equipment had been damaging the high areas between the wheelpaths. However, it is more likely that the traffic has tended to seal the surface of the wheelpaths creating higher contrast between them and the balance of the roadway. A close examination of the following two photos from the same area shows that there has been very little aggregate loss.

Photo 10: Wheelpath

Photo 11: Lane Centre

There was one other interesting observation on the Duke Point section. This was the contrast between areas that were almost always shaded versus those that were in the open. The following picture illustrates this.

Photo 12: Shaded Area

In comparison to the previous pictures the rock was quite exposed and the surface asphalt appears to have worn off. There does not seem to have been any aggregate loss.

Generally, the Duke Point section was in good condition a full eleven years after construction.

4.0 PAVEMENT DATA ANALYSIS

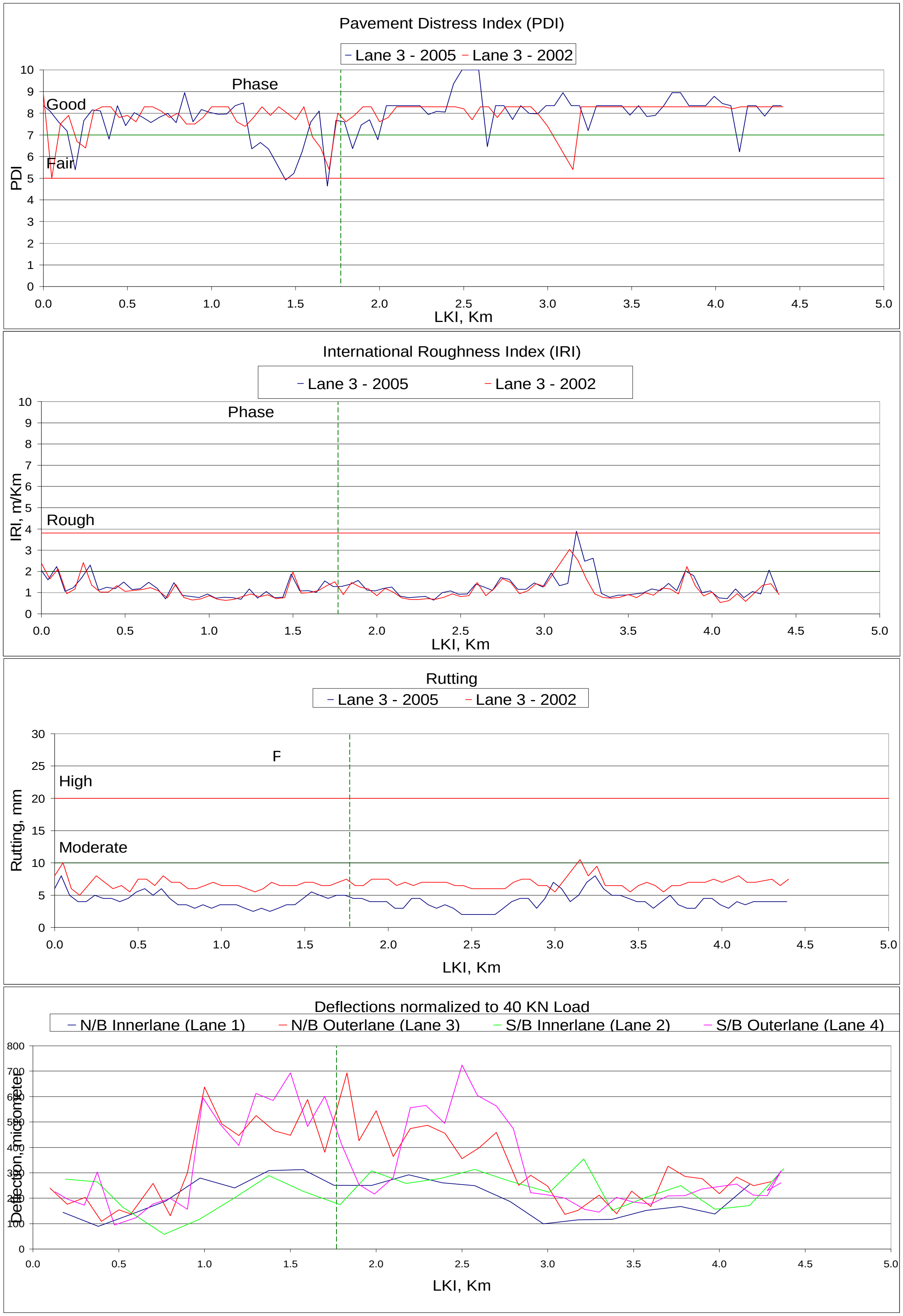
4.1 Pavement Management System (RPMS) Data

BC MoT Roadway Pavement Management System (RPMS) data was obtained from the Ministry for the years 1997, 1999, 2002 and 2005 for Highway 19A and for years 1995, 1997, 1999, 2001, 2003, 2005 and 2007 for Highway 19. The RPMS is BC MoT's pavement management system for their provincial highway network. Pavement Distress Index (PDI), Roughness (IRI), and Rutting data for the two most recent years were analysed.

Highway 19A – North Nanaimo

Plots of the RPMS data for Highway 19A are presented on the following page. Analysis of the RPMS data for lane 3 (northbound outer lane) for 2005 showed that the PDI ranged between 4.64 and ten with an overall average PDI of 7.87. The PDI index is dominated by pavement cracking. A PDI of ten represents a perfect condition, while zero represents the worst condition possible. A PDI above seven is considered *good*, between five and seven *fair*, and below five *poor*.

Figure 2: Roadway Pavement Management Data Highway 19A - North Nanaimo



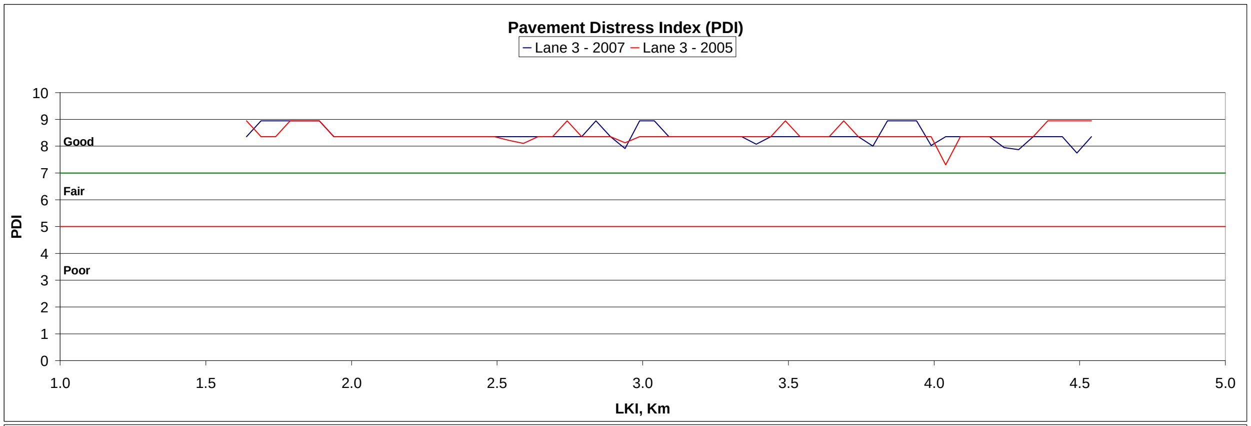
The roughness of lane 3 for 2005 as measured by the International Roughness Index ranged between 0.65 m/km and 3.89 m/km with an overall average of 1.24 m/km. IRI below two metres per kilometre is considered *good*, between two and 3.8 *moderate*, and above 3.8 *rough*. The average roughness for 2002 was 1.12 for both lane 3 and lane 4. The cumulative analysis of roughness data for 2005 (lane 3) showed that only seven percent of the pavement was found to have moderate roughness (3.8>IRI>2), while 93% of the pavement was found to have low roughness (IRI < two). The data analysis showed that there was no significant increase in roughness between 2002 and 2005.

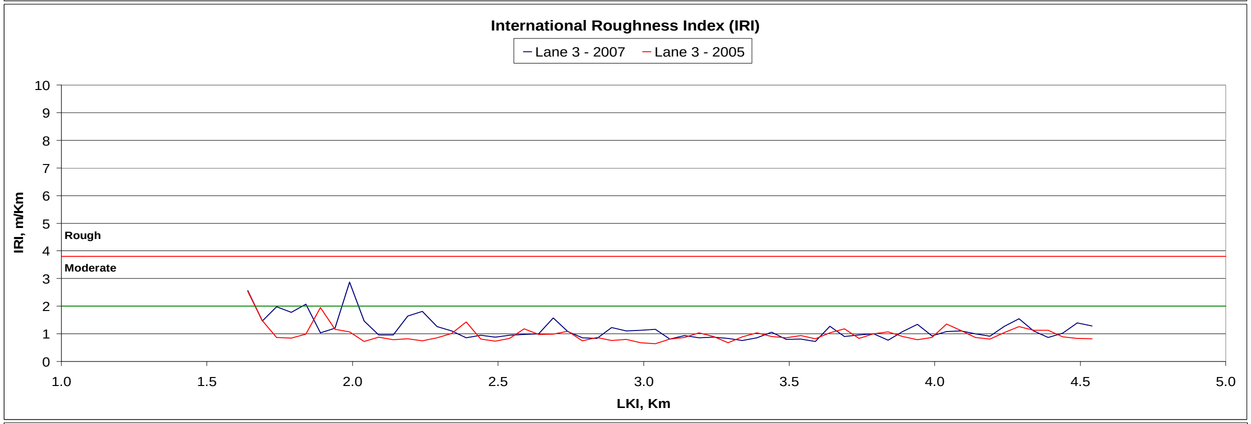
Rut depth of lane 3 for 2005 was found to be of low severity (rut depth < 10 mm) and ranged between two and eight millimetres with an overall average of four millimetres. This is consistent with the subjective visual observations.

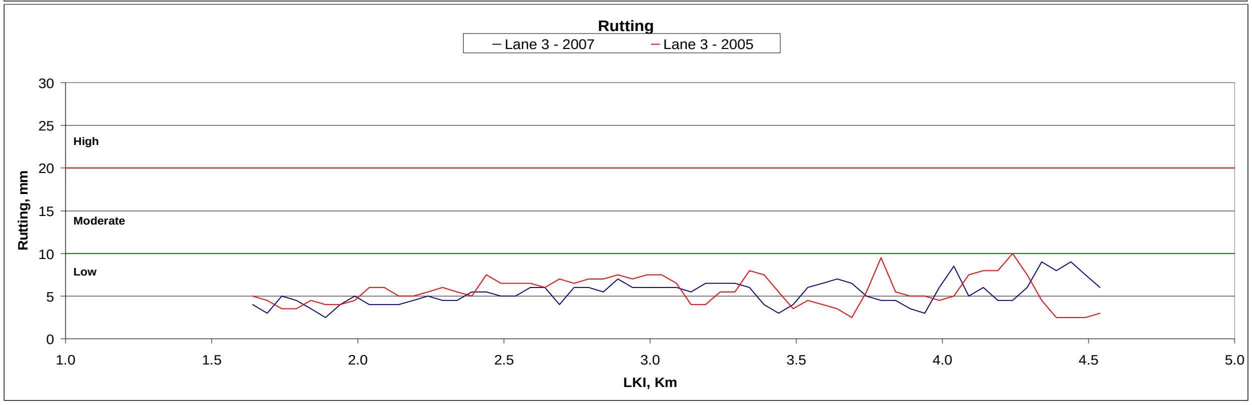
Highway 19 – Duke Point Highway

Pavement Distress Index (PDI), Roughness (IRI), and Rutting data for the Duke Point Highway for the years 2007 and 2005 are presented on the following two pages.

Figure 3: Roadway Pavement Management Data – Highway 19 – Duke Point







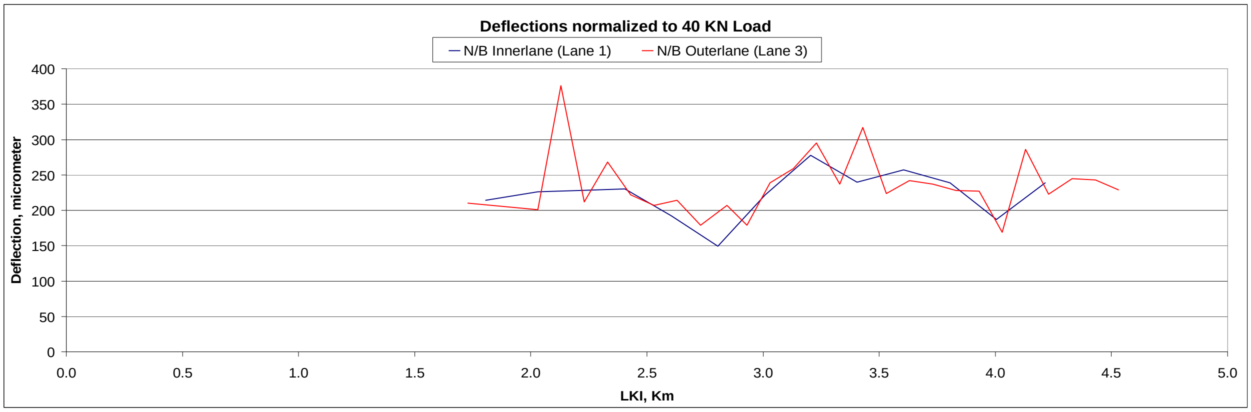
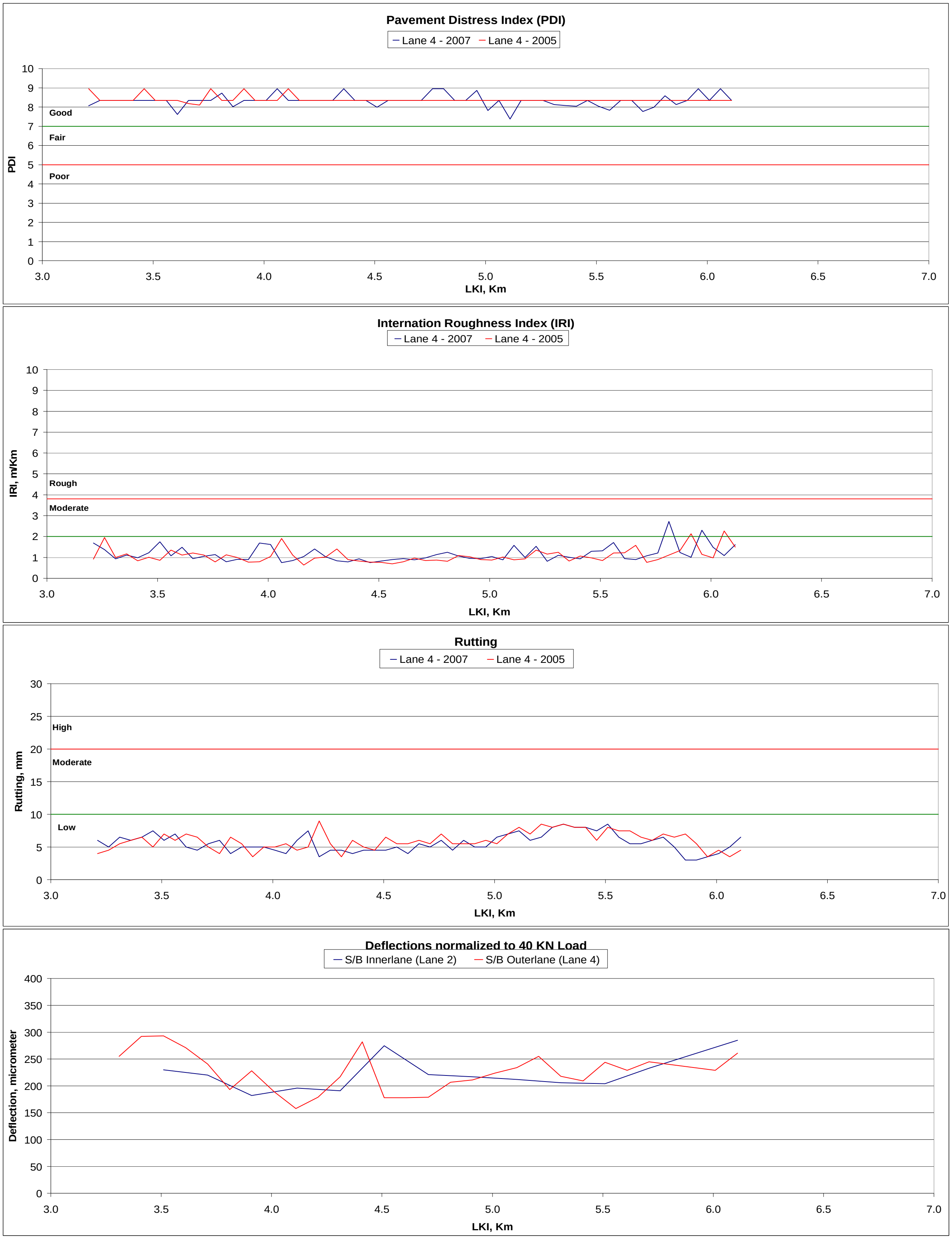


Figure 4: Roadway Pavement Management Data – Highway 19 – Duke Point



Analysis of the RPMS data for the year 2007 showed an average PDI of 8.41 for lane 3 (northbound outer lane) and 8.33 for lane 4 (southbound outer lane). The average PDI for 2005 was 8.43 for lane 3 and 8.39 for lane 4. The cumulative analysis of PDI data of lane 3 and lane 4 for 2007 showed that 100% of the pavement was in good condition (PDI > 7).

Roughness of lane 3 ranged from 0.73 m/km to 2.87 m/km. The average roughness was 1.17 m/km. It was found that 95% of lane 3 has low roughness (IRI < 2.0 m/km). The roughness of lane 4 ranged between 0.75 m/km and 2.72 m/km with an overall average of 1.16 m/km. Some 97% of lane 4 has found to have low roughness. The comparison of 2005 and 2007 roughness data showed no significant increase in pavement roughness.

Rut depth for lane 3 and lane 4 for 2007 was found to be of low severity (Rut depth < 10 mm).

4.2 PAVEMENT STRENGTH DATA

A Falling Weight Deflectometer (FWD) survey was carried out on 20 October, 2008[i]. The travel lanes were tested at approximately 100 m intervals for the outer lanes and 200 m intervals for the inner lanes. Resulting deflections and backcalculated layer modulus were calculated using DARWin 3.1 software. The pavement strength results are discussed in the following paragraphs.

Highway 19A – North Nanaimo

The normalized deflection values are presented on the above RPMS plots (Figure 2). The cumulative analysis of deflection data showed that the deflections in the inner lanes were smaller than the deflections in the outer lanes and therefore the inner lanes were stronger at the time of testing.

The backcalulated effective subgrade resilient modulus (M_r) values ranged from 30 to 150 MPa northbound and from 38 to 171 MPa southbound. The average backcalculated subgrade modulus was 93 MPa for lane 1; 63 MPa for lane 3; 90 MPa for lane 2; and 73 MPa for lane 4. Cumulative analysis of backcalculated subgrade modulus shows that the subgrade for the outer lanes is weaker than the subgrade for the inner lanes, which is in conformance with the higher deflections in the outer lanes from the cumulative analysis of deflection data.

The strength analysis indicated that there is a need to strengthen this roadway to achieve a new 20 year design fatigue life.

Highway 19 Duke Point

The normalized deflection values are presented on the above RPMS plots (Figures 3 and 4). A cumulative analysis of the deflection data showed higher deflections on the outer lanes than the inner lanes in both directions.

The backcalculated effective subgrade resilient modulus (M_r) values ranged from 56 to 274 MPa northbound and from 50 to 224 MPa southbound. The average backcalculated subgrade modulus was 146 MPa for lane 1; 139 MPa for lane 3; 136 MPa for lane 2; and 130 MPa for lane 4. The cumulative distribution of M_r shows a slightly more uniform subgrade modulus for the southbound outer lane (lane 2) than for the other lanes.

The strength test results indicate that this roadway is structurally adequate with no need for an overlay.

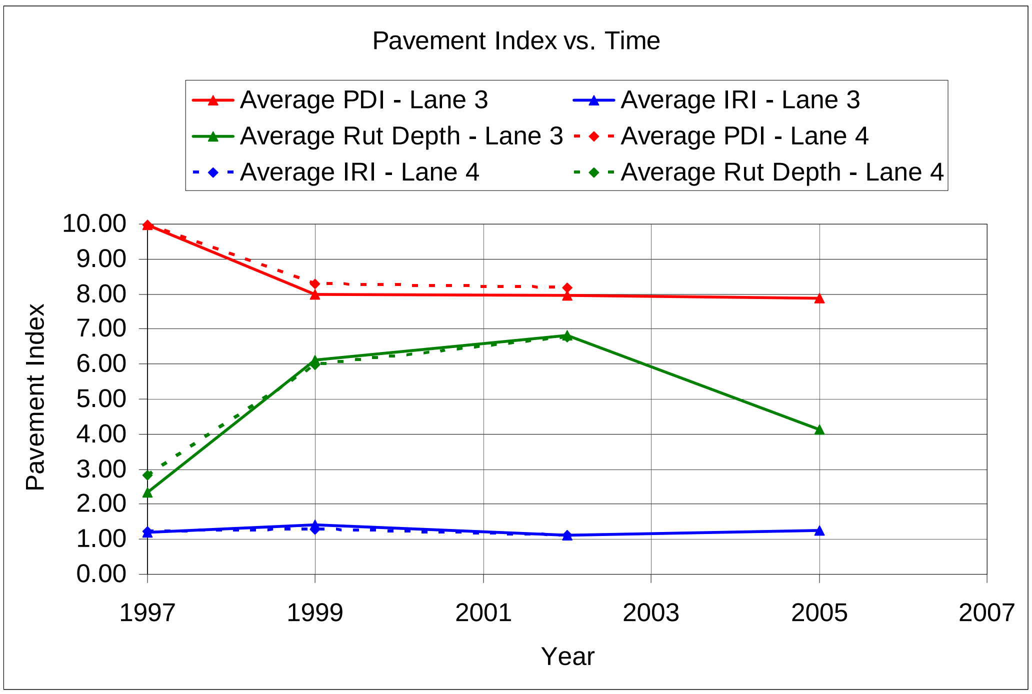
5.0 PAVEMENT PERFORMANCE

In order to assess change in pavement condition since construction, the RPMS data was plotted as a function of time.

Highway 19A North Nanaimo

The figure below shows the RPMS data vs. year starting in 1997. It shows that the average Pavement Distress Index (PDI) dropped from ten to eight during the first two years and has remained fairly constant thereafter.

Figure 5: RPMS vs. Time – Highway 19A

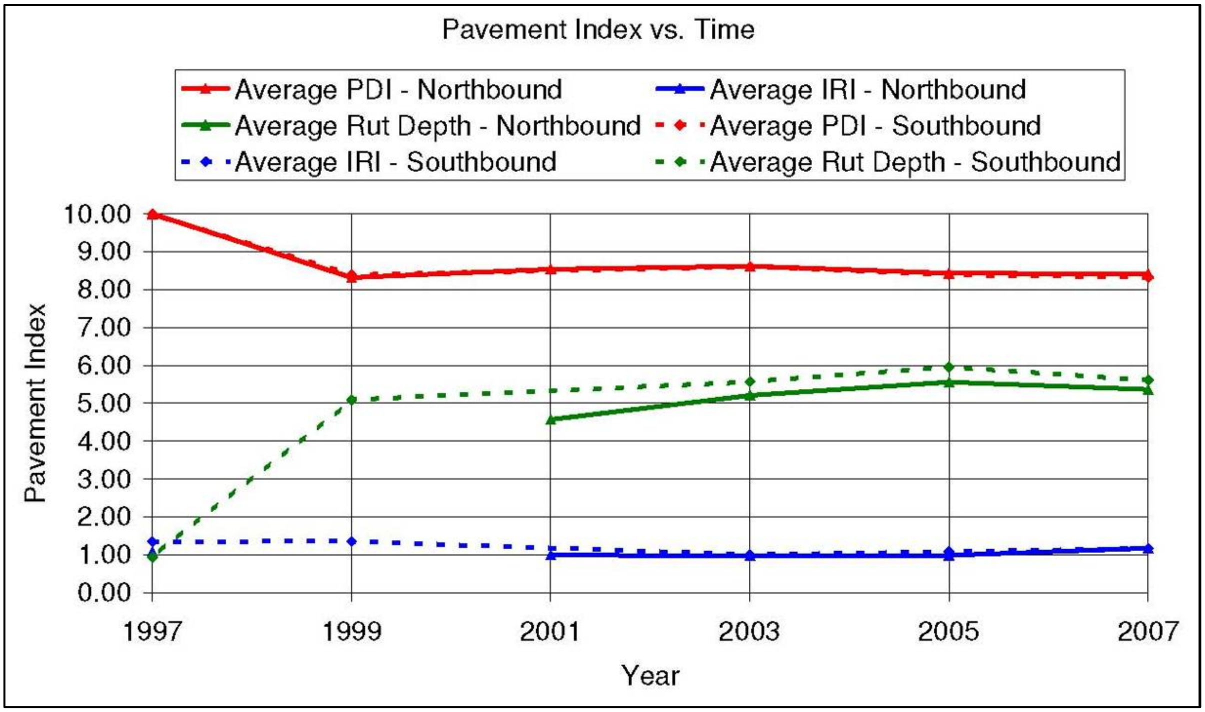


In the same time, rut depth increased from 2.3 mm in 1997, to 6.0 mm in 1999, then to 7.0 mm in 2002. There appears to have been a reduction in rut depth from 2002 to 2005. The apparent reduction is likely a function of the accuracy of the automated data collection system, the BC MoT acceptance criteria for automated rut data collection allows plus or minus three millimetres variation from manually collected rut measurements[ii]. The pavement condition of Phase 2 was found to be slightly better than Phase 1 (PDI higher, rut depth and roughness lower). However the roughness of both phases is quite the same after 2002.

Highway 19 Duke Point

The figure below shows the RPMS data vs. year from the time that the OGFC was placed. It shows that the PDI has dropped from ten to 8.3 during the first two years of operation of the highway after 1997 and has remained fairly constant thereafter.

Figure 6: RPMS vs. Time- Highway 19 - Duke Point



IRI appears to have remained fairly constant from 1997 to 2007 at between one and 1.5 m/km.

Rutting depth has increased from 1.0 to 5.0 mm from 1997 to 1999, and increased to 6.0 in 2005.

6.0 CORING, SKID RESISTANCE AND PERMEABILITY

Fieldwork was carried out in March 2009 and consisted of coring, measurement of skid resistance, and measurement of in place permeability. The test areas were divided up into Highway 19A Phase 1 paved in 1995, Highway 19A Phase 2 paved in 1996, and Duke Point Highway paved in 1997. Three test locations were selected in each of the three segments. The following figure presents the test locations for Highway 19A, north Nanaimo.

Figure 7: Highway 19A North Nanaimo Test Locations

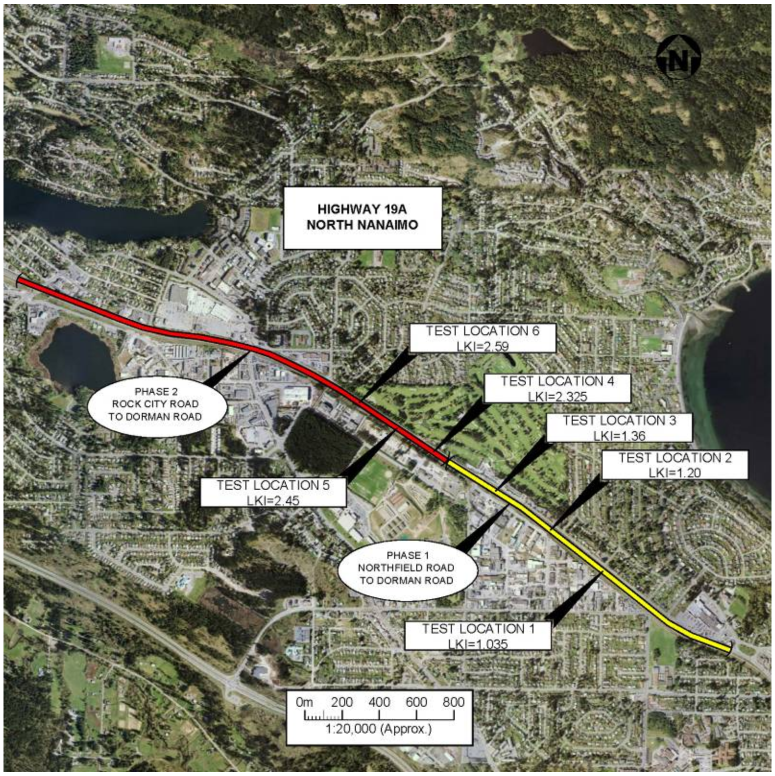
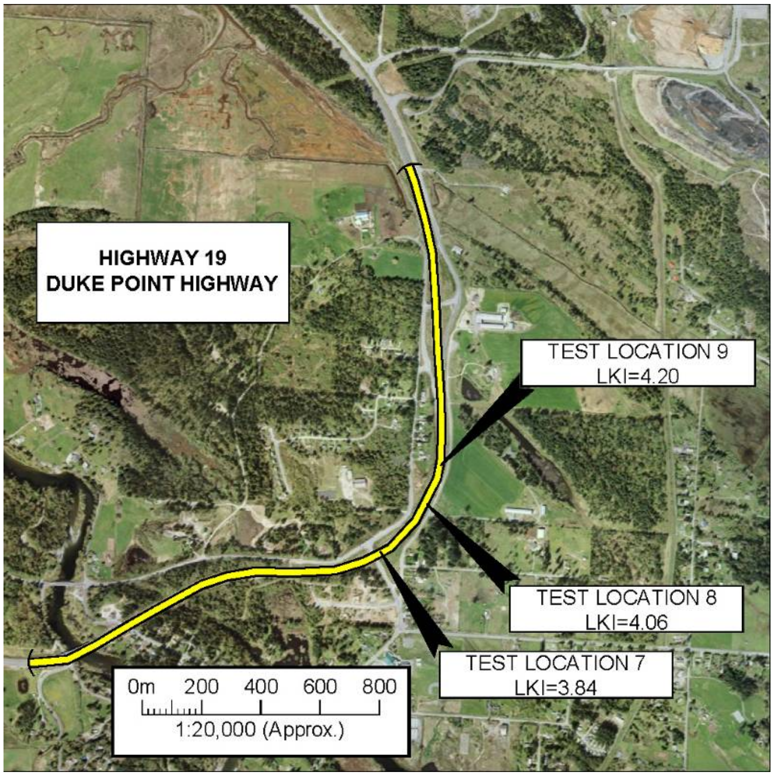


Figure 8: Highway 19 Duke Point Test Locations



The work on Highway 19A was done in the southbound slow lane and the work on Highway 19 was done in the eastbound slow lane. These locations were selected in order to have minimal interference with traffic. The results are presented in the following table (Table 3).

Table 3: Summary of Cores and Test Results

Test Area Number	Core and Test Location	Location - LKI (km) [2]	Total Core Thickness (mm)	OGFC Thickness (mm)	Skid Resistance (BPN)		Permeability (cm/s)[1]		Stripping
					Shoulder	Wheelpath	Shoulder	Wheelpath	
1	Southbound Lane	1.04	79	17	99	97		1.7E-05	NO
2	Southbound Lane	1.20	80	14	91	105	6.2E-04		NO
3	Southbound Lane	1.36	71	12	96	107		1.9E-05	NO
Average Phase 1			77	14	95	103	6.2E-04	1.8E-05	
4	Southbound Lane	2.32	110	31	92	103		9.2E-06	NO
5	Southbound Lane	2.45	112	30	80	88	3.8E-04		NO
6	Southbound Lane	2.59	80	30	82	62		1.6E-04	NO
Average Phase 2			101	30	85	84	3.8E-04	8.5E-05	
7	Northbound Lane	3.84	117	30	104	87		1.2E-04	NO
8	Northbound Lane	4.06	85	30	101	97	4.3E-04		NO
9	Northbound Lane	4.20	75	35	103	93		5.8E-05	NO
Average Duke Point			92	32	103	92	4.3E-04	9.0E-05	

NOTES: [1] Permeability values corrected to 20ºC using Florida DOT corrections for viscosity

[2] LKI represents Landmark Kilometer Inventory, a BC MoT linear referencing system.

6.1 Coring

The cores were taken to allow measurement of the asphalt layers, observation of stripping, and observation of the void structure to check for accumulation of sand and silt.

The thickness of the OGFC layer was less than the design thickness for all cores. Highway 19A Phase 1 averaged 14 mm; Phase 2 - 30 mm, and Highway 19 Duke Point 32 mm. However, three cores per segment are likely too few to draw a conclusion regarding the representative OGFC thicknesses for the projects.

The cores were cut at the horizontal joint between the OGFC layer and the rest of the core. The OGFC portion was then placed in an oven for one hour at 60° C. The cores were broken in half and examined with a magnifying glass to try to determine if there was any visible evidence of asphalt stripping. None was found, but the void structures in all of the cores were observed to be filled with dirt and fines. As can be seen in the below photograph below the amount of dirt was significant and would seriously impede water flow. This was in evidence for all of the cores that were taken.

Photo 13: Side View of Core Showing Voids Filled with Debris

An extraction test was done to determine the approximate quantity of dirt being held by the interconnected voids. A comparison of the fines content from the extraction to that of the original mix design indicated that the core contained approximately five to seven percent fines debris by dry mass.

6.2 Skid Resistance

The skid resistance tests were conducted in accordance with ASTM E303 *Standard Test Method for Measuring Surface Frictional Properties using the British Pendulum Tester*. The units for the measurements are British Pendulum Number or BPN. This is always measured in a wet condition and a higher number indicates increased resistance to skidding. This method was chosen because there had been measurements taken using this method in the past and a direct comparison could be made.

The results indicated relatively better skid resistance for the Highway 19 Duke Point segment when compared with Highway 19A north Nanaimo. The results for Highway 19A Phase 1 averaged 95 for the shoulder and 103 for the wheelpaths. The results for Phase 2 of Highway 19A averaged 85 for the shoulder and 84 for the wheelpaths. The results for Highway 19 Duke Point averaged 103 for the shoulder and 92 for the wheelpaths.

Bishop and Oliver[iii] reported on the original skid resistance of the Nanaimo OGFC pavements. They showed average British Pendulum Numbers for the OGFC of 67 in 1995, and 72 in 1997. They indicated an average BPN of 61 for a nearby conventional pavement.

The present results indicate a significant improvement in skid resistance of the OGFC pavement over time, and a significant increase over the conventional dense graded asphalt concrete pavements.

6.3 Permeability

The permeability testing was conducted with apparatus meeting the requirements of Field Permeameter No. 3 described in National Center for Asphalt Technology (NCAT) Report 99-01[iv]. The apparatus is shown in the following photograph:

Photo 14: Coring and Permeability Testing

The permeability testing showed relatively low permeability in each location. The results for the shoulders ranged from 2.2×10^{-4} to 3.8×10^{-4} cm/s. The results for the wheelpaths ranged from 1.0×10^{-4} to 5.9×10^{-6}.

Permeability testing had not been carried out when these pavements were new. Schaus, Tighe, and Uzarowski[v] reported average coefficients of permeability of 0.99 to 1.0 cm/s for tests completed on new OGFC. The void ratio and other properties of the mixes will not be identical to the Nanaimo pavements, however, the contrast in permeability is four to six orders of magnitude.

In most cases the permeability for the shoulder exceeded the permeability for the wheelpath. The ratio of permeability from the wheelpaths to the shoulders averaged six. This is expected from the amount of dirt clogging the cores taken in the wheelpaths.

The permeability testing was very revealing of the condition of the void system in the OGFC. There was a feeling that during wet weather any dirt in the voids would be washed out by the pumping action of vehicle tires passing over the pavement. This does not appear to be the case since the permeability of the aged pavement is four to six orders of magnitude less than that of a new pavement, and the permeability in the wheelpaths is generally six times lower than outside the wheelpaths.

7.0 SOUND TEST RESULTS

A study began in 1995 to assess the noise reduction performance of the Open Graded Friction Course (OGFC) pavement that had been applied that year on Highway 19A (current designation) in north Nanaimo and track its performance over a three-year period. The tracking period was subsequently extended to a fourth year. A second study was initiated in 1997 to assess the benefit of a second of OGFC pavement applied on the Duke Point Highway and to reassess it in September of 1999.

The measurement method was a *pass-by* type configuration with measurement stations set up next to the roadway. The noise measurements have all been made with a Larson-Davis Model 820 Community Noise Analyzer. This precision instrument complies with ANSI S1.4 [1983] for Type 1 Sound Level Meters and is capable of sampling ambient noise levels several times per second. The LD 820 was calibrated both before and after each noise measurement period using a Larson-Davis Type CA200 Precision Acoustic Calibrator. Monitoring occasions were selected to provide suitable clear, dry weather.

The key noise metric measured by the LD 820 is the Equivalent Sound Level, or L_{eq}. The L_{eq} is an energy-based average sound level that can be computed over any desired time period. It is expressed in units of A-weighted decibels, or dBA. When taken over a 24-hour day, the outcome is the $L_{eq}(24)$ which is the noise metric used in the BC MoT's Revised Noise Policy of 1993.

At the outset of the noise study, prior to the OGFC pavement being applied on Highway 19A, "pre-OGFC" baseline noise measurements were made at the demonstration section as well as at a "control section" of highway having standard asphalt pavement and being located just north of the OGFC section. The control section had nominally the same traffic characteristics and geometry as the OGFC test section. However, less than a year later, the control section was also paved with OGFC. With the loss of this side-by-side control section, OGFC noise measurements in subsequent years were limited to a linear study of noise level variations over time within the initial OGFC test section. It was therefore necessary to make appropriate corrections to the measured traffic noise levels for any observed year-over-year variations in traffic volumes, heavy truck mix and average vehicle speed. One particularly significant variation in traffic conditions involved the transfer of a significant amount of traffic from the route of the test section to the Nanaimo Parkway following the latter's opening on May 31, 1997.

All noise measurements at the OGFC demonstration section on Highway 19A have been made at the same location. The measurement position was on a gentle, grassy slope to the west of the highway, between it and the E&N Railway line and approximately at the middle of the 1.6 km long Phase 1 OGFC test section. The LD 820 was located 15 m from the centreline of the two southbound lanes with its microphone mounted on a tripod at an elevation of 1.5 m above the ground. A typical sound measurement setup is shown in the following photograph:

Photo 15: Sound Measurement Set-up

The initial measurements of Duke Point Highway traffic noise were made on July 28, 1997 shortly after the highway opened. On September 14, 1999 these measurements were repeated. In both of these earlier cases and in the recent 11-year follow-up, the procedure involved simultaneous noise measurements at two locations, both located on the north side of the highway, 15 m from the centre of the two westbound lanes. The longer, eastern portion of the Duke Point Highway had been paved with OGFC while the western portion had been paved with standard dense graded asphalt. It was therefore possible to carry out simultaneous “side-by-side” noise measurements on the two types of pavement. Since there are no entrances or exits to or from the highway between these two noise monitoring sites, traffic volumes and truck mixes past both sites are identical. Variations in average vehicle speeds can, however, occur. Both noise measurement sites are located on upward sloping ground, slightly overlooking the highway. In this way, any noise shielding effects of the concrete roadside barrier, which is present at both sites, were minimized.

The eleven-year follow-up noise measurements were conducted on September 29, 2008 and again on October 21, 2008. During both monitoring sessions, limited traffic counts were made, primarily to establish the heavy vehicle mix for comparison with those that existed in 1997 and 1999. Vehicle speed measurements were also made at both monitoring sites using the Bushnell radar gun. This was done so that, if necessary, the difference between the average traffic noise levels measured at the two sites could be adjusted, if necessary, to account for any observed difference in average vehicle speeds.

The results of these field measurements showed that the traffic noise reduction effect provided by the OGFC in north Nanaimo was approximately 4.8 dBA over the first two years prior to the opening of the Nanaimo Parkway in May 1997 and the resulting diversion of traffic from Highway 19 (the designation of which was changed to Highway 19A at that time). In the two subsequent measurement years (1998 and 1999), while it was then necessary to make adjustments for year-to-year variations in traffic volumes and speeds, the OGFC noise reduction effect on Highway 19A remained between 4.0 and 4.5 dBA. Side-by-side traffic noise measurements conducted in 1997 and 1999 on the new Duke Point Highway and involving both a section of OGFC and an adjacent one of conventional asphalt revealed OGFC-related traffic noise reductions of 4.8 and 4.7 dBA respectively. To put these pavement-related noise reductions in perspective, a five decibel reduction in the level of a given noise is typically perceived as representing approximately a 30% reduction in subjective loudness or noisiness.

The current study has involved the duplication of the pavement noise measurement of the 1990's after an elapsed time of nine years. At the time of the new measurements in the fall of 2008, the north Nanaimo OGFC section was 13 years old while the Duke Point Highway section was 11 years old.

The average traffic noise level measured on October 22, 2008 at the north Nanaimo site was 1.4 dBA lower than that measured 13 years before in September 1995 prior to the application of the OGFC. However, when the necessary traffic noise level adjustments are made to account for the lower traffic volume, average vehicle speed and heavy vehicle mix that now exist on this section of Highway 19A, the noise reduction benefit of the OGFC was found to have essentially disappeared. The residual noise reduction benefit of the OGFC as measured on the Duke Point Highway in September and October of 2008 was found (after adjusting for the five kilometres per hour higher vehicle average speed observed on the conventional asphalt section) to be approximately one decibel.

The following figures present the noise test data:

Figure 9: Traffic Noise Level Highway 19A - North Nanaimo

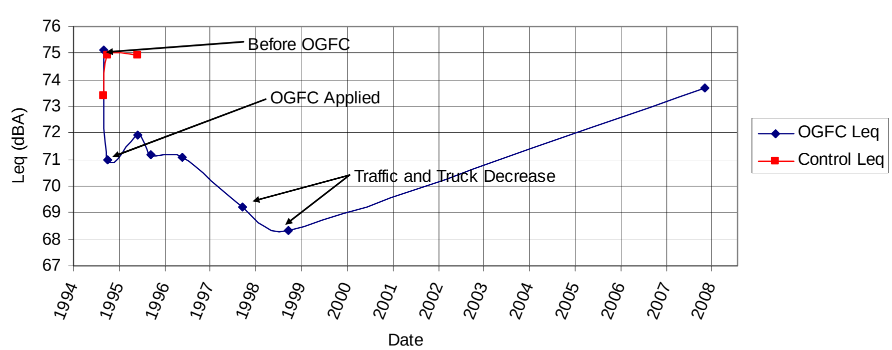
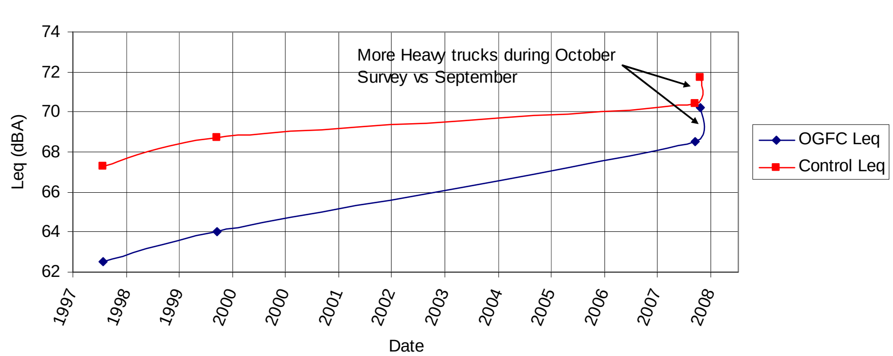


Figure 10: Traffic Noise Level Highway 19 - Duke Point



It may be concluded that, after 13 and 11 years of service respectively, the OGFC pavements applied on Highway 19 in north Nanaimo and on the Duke Point Highway demonstrate little or no residual noise reduction benefit. The primary reason for this loss of noise attenuation performance over time is believed to be the gradual clogging with dirt, sand and tire rubber debris, of the many interconnecting voids which characterize these open-graded pavements. It would therefore be of interest to explore the effects that cleaning this debris from the OGFC pavement could have on its noise reduction capacity.

8.0 DISCUSSION AND CONCLUSIONS

The open graded surface friction pavements have been observed after 12 to 14 years of service. The Highway 19 pavement is performing better than the Highway 19A pavement and this is expected since the Highway 19 OGFC was constructed as a new roadway, while the Highway 19A OGFC was placed on an old pavement as part of rehabilitation. The traffic on Highway 19 is also much lower than on Highway 19A.

Based on the Roadway Pavement Management System data, each of the segments is generally within what is considered an acceptable condition with respect to Pavement Deterioration Index, International Roughness Index, and rutting which are the main indexes of pavement performance for BC MoT.

Based on PDI, Highway 19A is mainly in good condition, with some areas in fair condition, and a trace of areas in poor condition. However, based on the visual condition assessment, there appears to be a more significant level of fatigue cracking present. Also, the structural assessment indicates that an overlay is warranted. It is expected that the deterioration of this pavement will accelerate. This is due to the overall strength of the pavement system and should not be considered to be a result of the use of the OGFC. The additional thickness of OGFC asphalt and the resulting smoother surface will have extended the pavement life.

Highway 19 is all in good condition based on the PDI data. The strength test data shows that it is structurally adequate. It is subjected to much less traffic that Highway 19A and was constructed as a new highway in 1997.

The OGFC surface for all segments appears to have retained considerable texture as evidenced by the photos, with the exception of the shaded area shown in Photo 12. Skid resistance test results show that the skid resistance has improved since the OGFC pavements were constructed.

It was observed that most of the surface deterioration of the OGFC surface had occurred during the first two years of operation in both highways according to the RPMS data.

The most significant result from the study was that the OGFC interconnected void system appears to have been uniformly clogged by silt and sand based on the permeability test results, and observation of the cores. The permeability of the OGFC appears to have dropped four to six orders of magnitude when compared to published results for a similar pavement. Winter maintenance in this area does not generally include the use of abrasives, so this material must have been tracked in by vehicles or blown in as dust. This has impacted several of the desirable properties of the OGFC surface including sound attenuation, and reduction of water spray in wet weather.

The OGFC in the shoulders appears to be six times more pervious than the wheelpaths. This appears to show that the void system has not been self cleaned from the water forced through the void system by wheels in wet weather.

Based on the noise monitoring conducted on October 22, 2008 on Highway 19A – north Nanaimo, the OGFC appears to have lost essentially all of its noise reduction benefit. While average noise levels on the west side of the highway are still slightly (1.4 dBA) lower than they were in 1995 before the OGFC was applied, when corrections are made to account for the decreases in hourly traffic volumes and average vehicle speeds that have occurred over the intervening 13 years, it must be concluded that the OGFC pavement is no longer quieter than the aged standard asphalt pavement that it replaced in 1995 and perhaps even a bit noisier.

October 2008 measurements have shown that the average traffic noise levels generated on the OGFC section on the Duke Point Highway are still approximately 1.7 dBA lower than those generated on conventional asphalt pavement. However, when a correction is applied to account for the fact that average vehicle speed was 5 kmph higher on the conventional section, the net residual OGFC effect is likely closer to one decibel. The discovery of a slightly greater residual OGFC effect on the Duke Point Highway than on Highway 19A would appear to support other findings (such as those of the Danish Road Directorate[vi]), that higher speed roads are to some extent “self cleaning”. However, other factors could account for all or part of this difference. For instance, the Duke Point Highway OGFC was placed one to two years after the Highway 19A OGFC, and the traffic level on the Duke Point Highway is much lower than that of Highway 19A.

The reduction in the sound attenuation benefit of the OGFC pavement appears to be a result of clogging of the interconnected void system with debris (dirt, sand and tire particles).

The above discussion may explain how, over time, OGFC looses its noise reduction benefit relative to standard asphalt but it does not explain how OGFC could actually become noisier than conventional asphalt, if in fact this has been the case. It may be speculated, however, that the surface profile of aged OGFC pavement is rougher than similarly-aged conventional asphalt. If this should be the case, after the OGFC’s porosity is essentially lost due to clogging, its rougher rolling surface may create slightly more tire noise than conventional pavement. Ahammed and Tighevii found a correlation between Mean Texture Depth and pass-by noise levels for various pavement types. These findings tend to support the above reasoning that the aged OGFC pavement with clogged voids might attenuate sound less than conventional dense graded asphalt concrete because the OGFC surface is rougher. However, it is also possible, given the uncertainties involved in both the measurement of traffic noise levels and the prediction of the effects on noise levels of traffic parameter variations (volume, speed and truck mix), that the observed difference between the average noise levels created by traffic on the conventional asphalt and that created on the OGFC pavement is within the inherent error margins of such analyses and hence is not significant.

The Nanaimo OGFC developmental pavement trials have confirmed that, in coastal areas at least, OGFC pavements can have service lives under moderate traffic volumes that are comparable to those of conventional asphalt pavements. Worthwhile traffic noise reduction benefits of OGFC were observed to persist over the medium term (four years on Highway 19A, two years minimum on the Duke Point Highway). It then appears that after about four years past application, this benefit begins to erode, so that it is largely or totally lost within 11 to 13 years of installation, due principally to the clogging of the OGFC’s pores with debris.

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[i] Report to BC Ministry of Transportation and Infrastructure, *FWD Strength Testing and Analysis, Highway 19 & 19A, Nanaimo, BC*, EBA Engineering, EBA File: V13101160.013, November 2008

[ii] *Pavement Surface Condition Rating Manual*, Third Edition, May, 2009

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[iii] *Open Graded Friction Course Pavements in British Columbia,* Bishop, Michael C., Oliver, Michael F., Proceedings of the Annual Conference- Canadian Technical Asphalt Association, 2001, ISSU 46, Pages 99-120.

[iv] *Permeability of Superpave Mixtures: Evaluation of Field Permeameters*, L. Allen Cooley Jr., National Center for Asphalt Technology, Auburn University, Alabama, NCAT Report 99-01, February 1999

[v] *Evaluation of Porous Asphalt Pavement Technology for a Canadian Climate*, Schaus, L. Tighe, S. Uzarowski, L., Proceedings of the Annual Conference- Canadian Technical Asphalt Association, 2007, ISSU 52, pages 349-372

[vi] *Technical Performance and Long-Term Noise Reduction of Porous Asphalt Pavements*, Jørn Raaberg, Bjarne Schmidt, and Hans Bendtsen, September 2001, Road Directorate, Danish Road Institute

[vii] *Asphalt Pavement Surface Texture, Friction, and Noise: Findings from CPATT Quiet Pavement Research*, Ahammed, M., Tighe, S. - Proceedings of the Annual Conference- Canadian Technical Asphalt Association, 2008, ISSU 53