Fifteen Years Performance of a Concrete Pavement Rehabilitation Project in Canada

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Abstract

The Ministry of Transportation of Ontario (MTO) undertook the rehabilitation of an exposed concrete pavement exhibiting various distress manifestations in the summer of 1989. Highway 126 in Southwestern Ontario is a four-lane divided arterial with 22,000 AADT and 9.6% commercial traffic in year 2000. The existing pavement, originally constructed in 1963, consisted of 230 mm mesh reinforced Portland Cement Concrete (PCC) pavement with dowelled joints at a spacing of 21.3 m. The rehabilitation strategy selected for southbound lanes was a 180 mm thick plain jointed unbonded PCC overlay to address the severe 'D' cracking and spalling at all the joints and cracks. The rehabilitation of the northbound lanes, which had experienced moderate deterioration, consisted of using the latest concrete pavement rehabilitation (CPR) techniques, material specifications and construction methods, which included full depth repair, partial depth repair, diamond grinding and joint sealant replacement. This paper will discuss the evaluation of this rehabilitated pavement in terms of roughness measurements, frictional resistance measured, and pavement condition ratings. Overall, the fifteen-year performance of the rehabilitated concrete pavements has been good with acceptable levels of ride quality, frictional resistance and distress propagation.

Introduction

The traditional method of rehabilitating concrete pavements in Canada before 1989 was to remove the distressed concrete full depth or partial depth and replace it with asphaltic concrete. These asphaltic concrete repairs performed well in the short-term, but the long-term hot mix patches would begin to distort and crack resulting in a rough ride.

During the mid to late 1980's, significant advances in concrete pavement rehabilitation (CPR) techniques in North America were achieved allowing the restoration of rigid pavements with concrete. This maintained the integrity of the PCC slabs and the long-term benefits of concrete.

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Highway 126 was selected in the late 1980's to demonstrate various restoration techniques, including methods that had not previously been utilized in Canada. Construction was completed late in the summer of 1989. At the time of construction, this project utilized the most comprehensive CPR techniques available at the time. Rehabilitation techniques included an unbonded overlay on the southbound lanes; and partial and full depth repairs and diamond grinding on the northbound lanes.

The demonstration site is located on a 5 km urban arterial facility which has since been assumed by the City of London and is now known as Highbury Avenue. Highbury Avenue is located approximately 200 kilometres west of Toronto. It is a four-lane divided arterial, with an AADT of 22,000, and 9.6% commercial vehicles in year 2000. According to the year 2000 traffic study data, the southbound commercial vehicle traffic is 11.3%, which is significantly higher than the northbound traffic of 7.9%.

Background

The original pavement on Highway 126 was constructed in 1963 with 225 mm of mesh reinforced PCC (JRCP) on 300 mm of granular base and subbase materials. The pavement was constructed with a 21.3 m joint spacing with dowel bars. The joints were sealed with preformed neoprene seals.

The pavement serviceability in 1989 varied dramatically between the northbound and southbound lanes. The pavement on the southbound lanes was in poor condition, with full and partial width transverse cracks in each slab, severe D-cracking, spalling of all joints throughout, and moderate pavement edge break-up.

The pavement condition of the northbound lanes was considered better than average considering the 26-year service life. The distresses on the northbound lanes consisted of approximately three transverse cracks per slab, with one being a severe working crack and two slight cracks, occasional spalling at joints and cracks, and a moderately polished surface. Typically the transverse contraction joints had seized, leading to the severe mid-slab working cracks. Minimal stepping was exhibited at the joints. The joint seals were approximately 85% intact, although there was significant bond failure between concrete and sealant throughout.

The variance in the 26-year pavement performance between the northbound lanes and southbound lanes was attributed to the different concrete aggregate types used in the concrete on the opposing lanes. The quarried aggregate used on the southbound lanes exhibited absorptive characteristics and was highly susceptible to D-cracking under freeze/thaw conditions. A pit run gravel source was used for the coarse aggregate on the northbound lanes which did not exhibit this D-cracking performance characteristic.

Design and Construction. The PCC rehabilitation techniques utilized on Highway 126 incorporated the latest designs and construction procedures. Three rehabilitation strategies were considered:

- crack and seat with an asphalt overlay;
- repair of the PCC pavement with hot mix and a multiple lift asphalt overlay;
- placement of an unbonded concrete overlay on the southbound lanes; and full depth repairs and spall repairs with concrete, diamond grinding on the northbound lanes

The estimated initial construction costs for the three options varied from \$1.5 to \$1.8 million, however, the total life cycle costs over 30 years were similar. The option of an unbonded concrete overlay, and concrete repairs with PCC, diamond grinding and was selected, as it best addressed the actual performance deficiencies exhibited by the pavement and provided the longest anticipated service life.

The design of the unbonded concrete overlay on the southbound lanes included the placement of a 20 mm asphaltic sand hot mix bondbreaker over the existing pavement surface. Then overlain by a 180 mm thick unreinforced plain jointed concrete pavement. Slipforming of the concrete overlay used revised specifications based on experience gained on an earlier MTO demonstration project on Highway 3N, which placed and monitored four types of experimental concrete pavements (Kazmierowski et al. 1985, Kazmierowski and Bradbury 1993). Skewed randomized joints were utilized with no effort made to avoid matching the underlying joints.

The design of the full depth repairs on the northbound lanes included a minimum repair width of 2.0 m. The dowel bars were epoxy coated and inserted into drilled holes spaced at 300 mm on both transverse faces. Also, the existing concrete had to be removed carefully not to disturb the underlying granular base.

The partial depth repair on the northbound lanes required sawcutting the perimeter to a depth of 50 mm, removal of loose concrete, and sandblasting of the final exposed surfaces. A cement paste was applied immediately prior to concrete placement. The designs used are fully detailed in (Kazmierowski et al. 1991).

In the construction stage, there were some initial difficulties with the removal of the existing concrete for the full depth repairs without disturbing the granular base. Once this was resolved, the construction of the concrete repairs, diamond grinding, and the unbonded overlay went smoothly. Details of the construction procedure and material testing have been published in the earlier report (Kazmierowski et al. 1991).

In order to maintain the integrity of pavement, routine maintenance on the rehabilitated concrete pavement was required. Resealing of cracks and joints were performed on the southbound lanes in 1997, in year 1995 and 2000 on the northbound lanes. Table 1 below indicates the remedial work carried on both the southbound and northbound lanes after the last major rehabilitation in 1989:

Table 1. Summary of Remedial Works Performed after 1989

Year	Location	Types & Quantities of Repair
1994	Northbound lanes	• Full depth repair - 800 m^2 • Partial depth repair - 35 m^2
1995	Northbound lanes	• Resealing of joints and cracks - 375 m
1997	Southbound lanes	• Full depth repair – 535 m^2 • Partial depth repair 10 m^2 • Resealing of joints and cracks – 110 m
2000	Northbound lanes	• Full depth repair – 1300 m^2 • Partial depth repair – 20 m^2 • Resealing of joints and cracks – 810 m

Pavement Distress Survey

The visible distresses in the rehabilitated pavement surface were surveyed using the ministry's Rigid Pavement Evaluation Procedure (Chong et al. 1995) to provide an overall indication of pavement performance. For the recent pavement distress survey carried out in 2004, a Pavement Condition Rating (PCR) measure was used to indicate the pavement performance. PCR is a composite pavement serviceability index ranging from 0 to 100; with the higher rating indicating a better pavement condition.

Pavement Performance on Southbound Lanes. In 1999, after ten years, there were minimal distresses identified in southbound lanes. The only distress observed in the southbound lanes was slight longitudinal cracking in 11 of the driving lane slabs. This can probably be attributed to inadequate longitudinal joint construction (i.e., delayed saw cutting) as the cracks were visible soon after construction. No transverse or reflection cracking has been observed, which correlates with other agencies' experience (Hall et al. 1989).

A recent survey of the southbound lanes in Fall 2004, after fifteen years, indicates the pavement is in good condition with a PCR of 70 in the driving lane, and 78 in the passing lane. Maintenance activities in southbound lanes consisting of routing and sealing of cracks and joints, as well as full depth and partial depth concrete repairs were carried out in 1997. There were a few moderate joint and crack spalling, localized slight faulting at the joints, and intermittent very severe pavement edge drop off. The longitudinal cracks identified earlier are now of moderate severity. In addition, a few severe reflection diagonal cracking and frequent moderate transverse cracks were identified. See Photo 1 below for the typical southbound lanes condition showing localized slight faulting at the joint.

Photo 1. Typical Unbonded Concrete Overlay Condition

Pavement Performance on Northbound Lanes. Northbound lanes concrete pavement restoration (CPR) consist of full depth repair and partial depth repair, as well as diamond grinding. In 1992, after three years, seven of the 266 full depth repairs were exhibiting a very slight longitudinal cracking in the right wheelpath of the driving lane. In a few locations, these cracks began to propagate into the adjacent slabs. No signs of movement or faulting of the repairs were observed. At that time, approximately half of the 75 partial depth repairs were exhibiting distress. The majority of the failures were in the driving lane and in the outside third of the slab.

In 1992, a limited investigation of selected partial depth repairs was undertaken. The investigation revealed that the partial depth repairs had extended to the level of the dowel bars, with associated joint movements resulting in rapid deterioration (cracking and spalling) occurring at the repairs. This reaffirms the accepted practice of not undertaking partial depth repairs for depths greater than one-third the slab depth or where shallow dowel bars/reinforcing meshes are encountered (Hall et al. 1992). Cores were taken through some slabs where the partial depth repairs did not exhibit any cracking. In these uncracked repairs, the partial depth repairs did not exceed a third of the slab depth and bond between the old and new concrete was excellent. In the deteriorated repairs, bond between the old and new concrete was also very good, indicating the weakness or spalling in the slab extended below the repair level. These findings confirms the importance of critical assessment of where partial depth repairs should be used.

In 1994, after five years, a detailed distress survey was also carried out indicating that further full depth repairs were required. A total of 96 areas requiring full depth repairs (including 13 of the original repairs) and 16 new partial depth repairs were identified. Pavement monitoring has revealed that where existing pavement joints are seized (non-working), and full depth repairs are constructed at mid-slab working cracks, future deterioration of secondary minor transverse cracks may occur. Consideration should be given to carrying out full depth repairs on these secondary cracks at the time of initial rehabilitation or scheduling them at a future date. The majority of the full depth repair locations required the minimum one lane width by 2.0 m long repair. The partial depth repairs were all one-half a square metre in size. An additional 46 cracks identified that need to be removed and replaced did not exist or were not wide enough to require full depth repairs at the time of original construction.

The City of London completed 96 full depth and 16 partial depth repairs along with limited joint and crack resealing in the summer of 1994. The repairs were completed in the same manner as the original repairs, except that the surface of the repairs was not diamond ground but tined longitudinally by hand, while the concrete was plastic.

In 1999, after ten years, the survey of the northbound lanes indicated that a few of the new repairs have cracked in the wheelpath of the driving lane, as happened with the original repairs. These cracks are slight in severity. There is also some slight spalling of the repairs and original concrete at the joints and cracks.

A recent survey of the northbound lanes in Fall 2004, after fifteen years, indicates the pavement is in fair to good condition, with a Pavement Condition Rating (PCR) of 63 in the driving lane, and 70 in the passing lane. Routing and sealing of cracks and joints, plus additional full depth and partial depth concrete repair was carried out in 1995 and early 2000. Most of the joints and cracks are performing well. There were a few moderate faulted joints and intermittent moderate joint and crack spalling. In addition, extensive severe pavement edge drop off was identified. See Photo 2 below showing the typical northbound lanes performance in 2004 adjacent to full depth repair.

Photo 2. Typical Northbound Lanes Pavement Condition

Roughness Measurements

Smoothness was measured just after construction, prior to opening to traffic, using the California Profilograph. The Portable Universal Roughness Device (PURD) was initially used for ongoing roughness monitoring on a yearly basis. In year 2001 the Automatic Road Analyzer (ARAN) replaced the PURD to collect International Roughness Index (IRI) based measurements.

The profilograph measurements were taken on the new concrete overlay in both wheelpaths of the driving and passing lanes. For details on the function and operation of a profilograph, refer to (Chojnacki 1985). The average profilograph reading (profile index) for the length of the project in the driving lanes was 175 mm/km and 169 mm/km in the passing lanes. These readings were taken prior to diamond grinding the 21 locations where the roadway did not meet the surface tolerance specifications of 3 mm in 3 m. For this demonstration project, the initial ride measured with the profilograph indicated that it met the smoothness requirements expected of a new exposed concrete pavement.

The PURD is a trailer-mounted, accelerometer-based mechanical measuring device operated at constant highway speed that uses the root mean square vertical acceleration of the trailer axle to measure roughness. PURD measurements are converted into a ride comfort index (RCI) using a transfer function for application in Ontario's Pavement Management System. RCI is based on a scale from 0-10, with 10 considered a smooth and pleasant ride.

Since 2001, the ARAN is used to collect roughness measurement. The ARAN consisted of a van traveling at highway speed, equipped with various testing and data logging systems. Rutting is measured (in mm) by a series of ultrasonic sensors that are mounted to an aluminum ‘smart’ bar at the front of the van. Roughness is measured within each wheelpath and converted into an International Roughness Index (IRI). IRI is a roughness measure with a scale of mm/m (or m/km). The IRI scale starts from 0 mm/m which represents absolute smoothness to an unlimited scale. The lower the IRI value, the smoother the pavement.

Testing for roughness was undertaken prior to rehabilitation, after rehabilitation just prior to opening to traffic, and on an annual basis since. For ease of comparison and consistency, all roughness measurements previously collected using PURD are converted from RCI to IRI.

Roughness Measurement on Southbound Lanes. The unbonded overlay rehabilitation on the southbound lanes in 1989 resulted in an improvement in roughness (IRI) from 3.89 to 1.66 and 2.79 to 1.60 in the driving and passing lanes respectively; see Figure 1. The ride quality levels (smooth, comfortable, uncomfortable) indicated on the graph are converted based on the established ride comfort index (RCI).

The newly tined surface texture on the unbonded overlay was very aggressive and had contributed to some of the initial roughness. The roughness in both the passing and driving lanes in the southbound direction demonstrate fairly constant IRI values from 1990 to 1995, with an average IRI of 1.85 and 1.72 in the driving and passing lane respectively.

The IRI values between 1996 to 2000 appear elevated probably as a result of an erroneous transfer function used in converting PURD measurements into IRI values. The conversion was based on limited correlation data. The surveys from 2001 to 2004 indicates the roughness remained fairly constant over these years, with an average IRI of 2.60 and 2.19 for the driving and passing lane respectively. Overall, the southbound lanes are in the comfortable category, indicating that after 15 years, the unbonded concrete overlay is performing at an acceptable level, well within anticipated life cycle parameters.

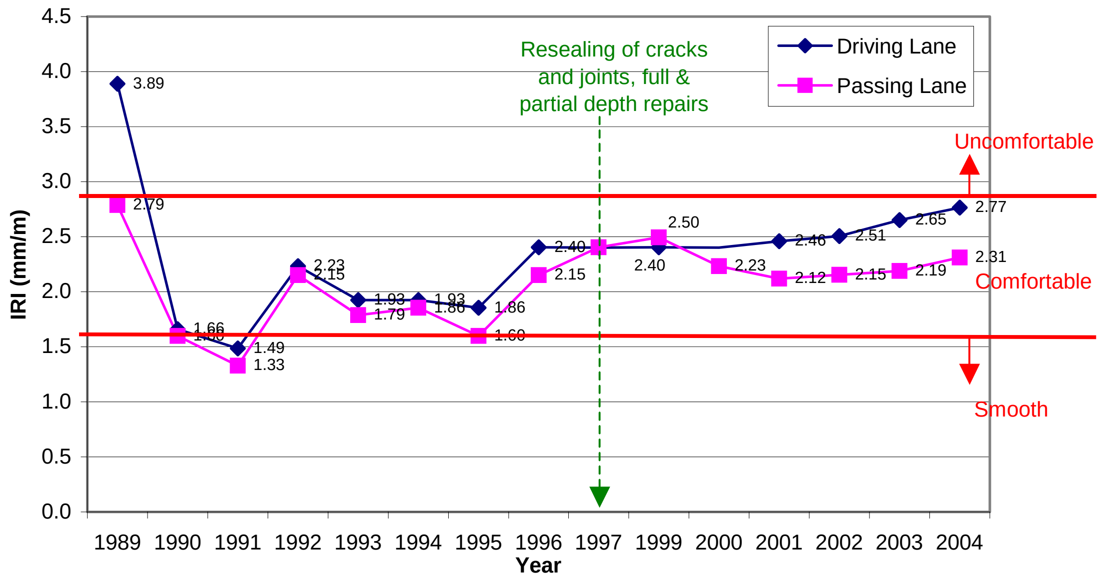

Figure 1. Summary of IRI Values in Southbound Lanes.

Roughness Measurement Northbound Lanes. Roughness in the northbound lanes improved significantly after the concrete pavement surface was diamond ground in 1989, with an improvement in IRI from initially 3.89 to 0.95 and 2.59 to 0.95 in the driving and passing lanes respectively, see Figure 2. The ride data for the diamond ground surface indicates that the initial roughness values deteriorated to an approximate value of 1.8 after two years and has remained fairly constant over the following three years. These IRI values are in the comfortable range of the ride categories.

Again, due to the incorrectly correlated transfer function used to convert PURD data to RCI coupled with several additional cycles of remedial work, resulted in the survey data from 1996 to 2000 being out of trend.

The ARAN surveys from 2001 to 2004 indicates the roughness remained constant over these years, with an average IRI of 2.32 and 1.83 for the driving and passing lane respectively. Overall, the northbound lanes rehabilitated using full CPR techniques are still performing within the comfortable category after 15 years of service.

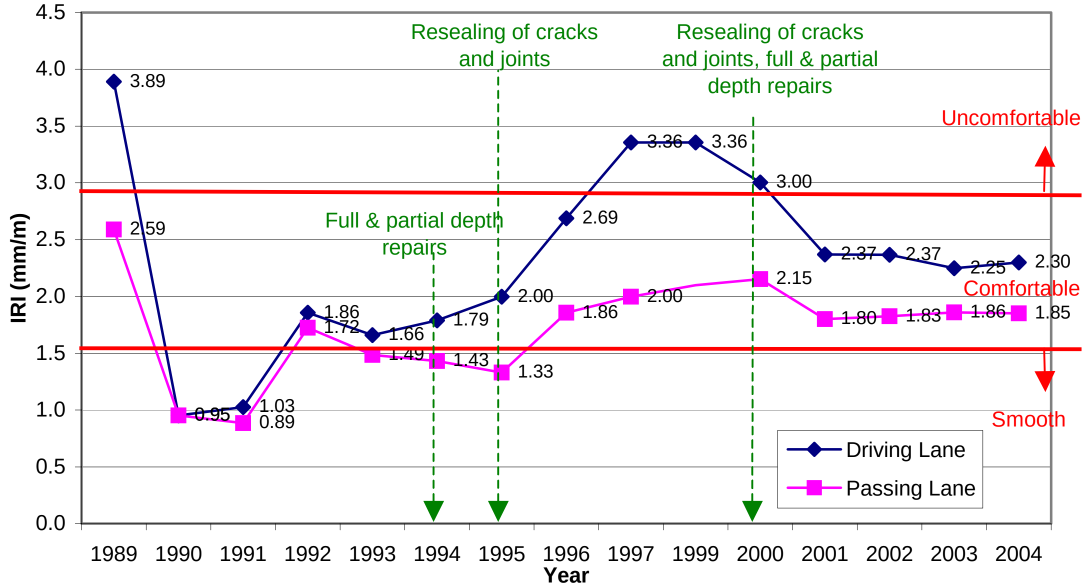

Figure 2. Summary of IRI Values in Northbound Lanes.

Skid Testing

The skid resistance of the pavement was measured with a brake-force trailer conforming to ASTM Standard E274. The measured friction force is described as a skid number (SN) at a specific speed; for example, SN 80 is the skid number at 80 km/h. A skid number is the average coefficient of friction across the test interval with a practical range from 10 to 60 (higher the better).

Skid Testing on Southbound Lanes. The skid numbers in the southbound lanes increased from SN 80 of 18 (driving lane) and 29 (passing lane) to an average SN 80 of 55 for both lanes immediately after the overlay in 1989. See Figure 3 for an annual summary of the skid resistance in the southbound lanes. The skid resistance has stayed fairly constant since construction with the driving lanes ranging from 31 to 39 (average of 34) and the passing lanes ranging from 37 to 45 (average of 41).

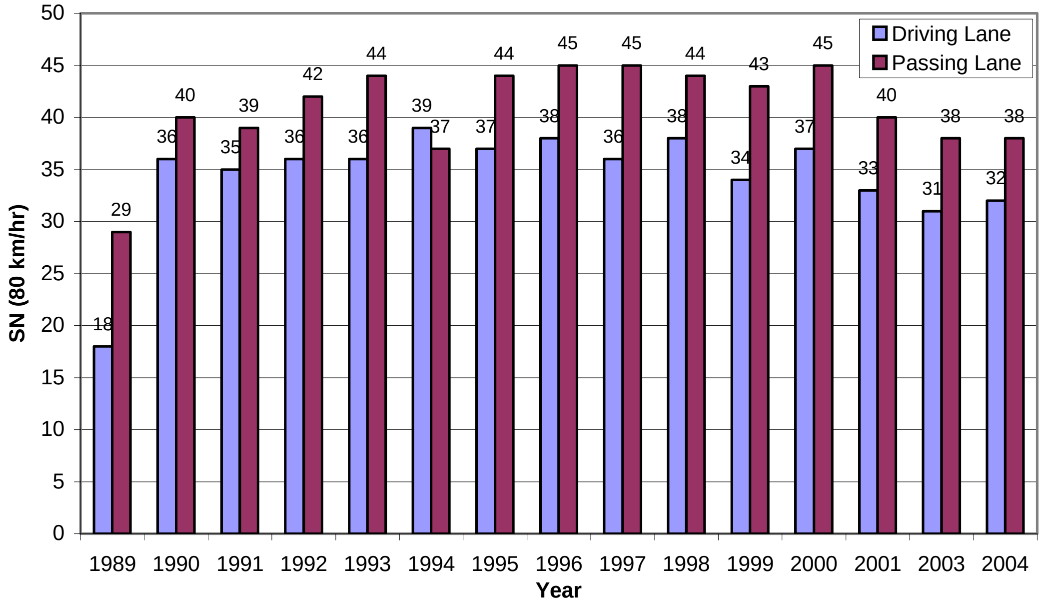

Figure 3. Annual Summary of Skid Resistance in Southbound Lanes.

Skid Testing on Northbound Lanes. For the diamond ground surface on the northbound lanes, the skid numbers improved immediately after construction in 1989, from an average of 25 to 48 and 35 to 49 for the driving and passing lanes respectively. Skid numbers of this magnitude indicate a good friction factor and are similar to numbers obtained on premium hot mix surface courses in Ontario (Ryell et al. 1978). Over the last fifteen years, the skid resistance appears to be consistent in the range of 24 to 33 (average of 27) for driving lane, range of 30 to 39 (average of 35) for the passing lane, see Figure 4.

These average values are approximately 6 units lower than the results of the tined and burlap texture on the southbound lanes. The fractured ridges in the ground surface have worn away fairly quickly under high traffic and heavy winter maintenance activities. The micro-texture created with diamond grinding does not appear to be as durable as that created by the burlap drag time texture in the southbound lanes, resulting in an earlier reduction in measured frictional properties.

Sound Level Measurements

Sound level measurements were undertaken prior to the rehabilitation to provide data for future projects. The procedure used to measure pavement noise on this project was to measure noise from individual vehicle pass-bys at a 15 m distance from the travelled lane. The burlap drag and transversely tined concrete surface on southbound lanes was approximately 1 dBA noisier than the old worn concrete and MTO sample. For further details on the sound measurements see (Kazmierowski et al. 1991).

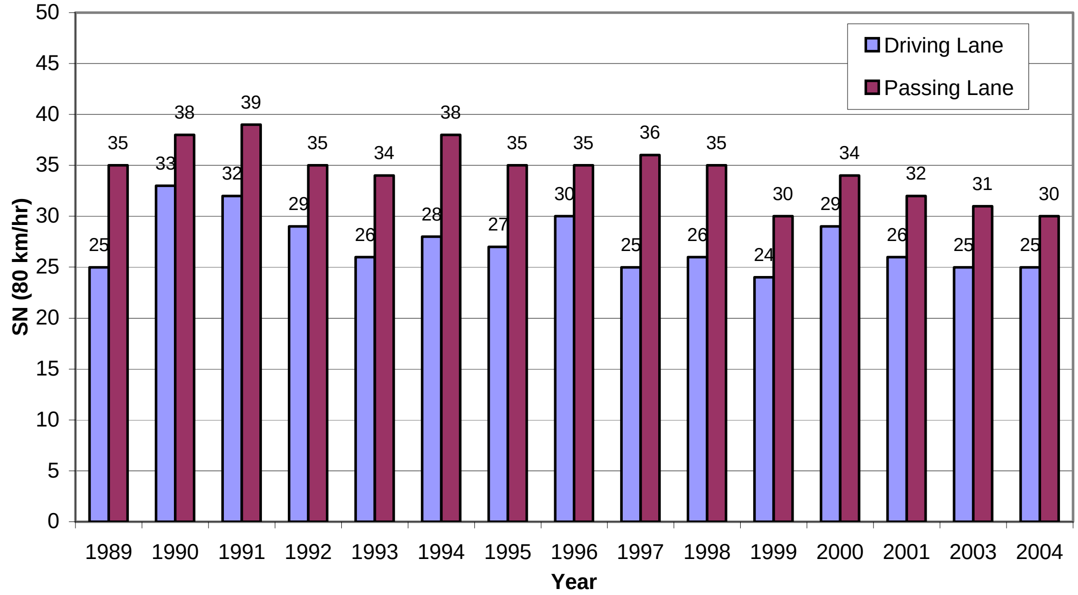

Figure 4. Summary of Skid Resistance in Northbound Lanes

The diamond ground surface was only 1 dBA louder than an open graded hot mix asphalt friction course, which is used for its low noise characteristics and good skid resistance. The diamond ground surface on northbound lanes was approximately 1 dBA quieter than an old worn concrete surface and the MTO sample. The MTO sample obtained from (Jung et al. 1985) was derived by testing several hundred vehicles on a variety of pavement surfaces, predominantly asphaltic concrete, across the province.

Conclusions & Recommendations

Monitoring the performance of the pavement over the last fifteen years has indicated the concrete pavement rehabilitation techniques used are performing acceptable on both the northbound (diamond grinding, full and partial depth repairs) and southbound lanes (180 mm unbonded overlay). The following chart (Table 2) is a summary indicating the performance of the pavement after fifteen years of service:

Table 2. Summary of Pavement Performance after 15 Years

PERFORMANCE AT YEAR 15	SOUTHBOUND	NORTHBOUND
Distresses	• Few, moderate joint & crack spalls • Intermittent, slight faulting along the joints • Frequent, moderate transverse cracking • Frequent pavement edge drop off (25 to 50 mm)	• Intermittent, moderate joint and crack spalls • Few, moderate faulting along the joints • Extensive, edge of pavement drop off (25 mm)
PCR*	74	66.5
IRI*	2.5	2.1
Skid Number*	35	28
Ride Quality	Comfortable	Comfortable

* These values are the average of driving and passing lanes readings.

The PCR for the southbound lanes is higher than the northbound lanes; however, the corresponding IRI values show the opposite trend. The northbound lanes are experiencing greater distress features which have not been reflected in the ride.

According to a recent traffic study, the southbound truck traffic is 40% higher than the northbound traffic. However, the 15-year performance of the unbonded 180 mm thick overlay on the southbound lanes has been good with acceptable ride quality and skid resistance, and with some reflection cracking and slight faulting at the joints.

The northbound lanes, which were treated with diamond grinding, full depth and partial depth repairs are also performing well. The diamond grinding significantly restored ride quality of the old concrete pavement for the fifteen-year monitoring period. Initially, diamond grinding at the northbound lanes was also very effective in restoring pavement skid resistance. However, after a few years the beneficial effects of diamond grinding for skid resistance has dissipated. After 15 years of service, measured skid resistance of the diamond ground surface was approximately 6 units lower than that of the burlap drag/tined surface on the southbound lanes.

Pavement monitoring in the northbound lanes has revealed that where existing pavement joints are seized (non-working), and full depth repairs are constructed at mid-slab working cracks, on-going deterioration of secondary minor transverse cracks has occurred. Consideration should be given to carrying out additional full depth repairs at the time of initial rehabilitation or scheduling them at a future date. Visual observations of partial depth repairs on the northbound lanes, in conjunction with core analysis, confirms the importance of both construction quality and judicious selection of repair locations on the effectiveness and durability of partial depth concrete repairs. The partial depth repairs that extended to the level of dowel bars resulted in rapid deterioration (cracking and spalling) of the repairs.

Monitoring the long-term pavement performance, associated with each type of concrete pavement rehabilitation (CPR) technique, is crucial in order to obtain data on life cycle costing of CPR strategies. The serviceability of this concrete pavement after 15 years since major rehabilitation has been good, with acceptable levels of ride quality, frictional resistance and distress propagation. In conclusion, the concrete pavement restoration techniques utilized on this project has significantly extended the life of the pavement. The knowledge gained on this project has been successfully used on numerous rigid pavement rehabilitation contracts in Ontario.

Acknowledgement

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