CPR Techniques in Canada
- 18 Years of Performance Experience

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Abstract

In the summer of 1989, the Ministry of Transportation of Ontario (MTO) undertook the rehabilitation of an exposed concrete pavement exhibiting various distress manifestations. 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 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 of the highway in the northbound lanes consisted of using the latest concrete pavement restoration (CPR) techniques, material specifications and construction methods. The rehabilitation techniques included full depth repair, partial depth repair, diamond grinding and joint sealant replacement on the northbound lanes which had experienced moderate deterioration. The southbound lanes received a 180 mm thick plain jointed unbonded PCC overlay to address the severe 'D' cracking and spalling at all the joints and cracks. This paper will discuss the eighteen-year evaluation of this rehabilitated pavement in terms of smoothness measurement using California Profilograph, roughness measurements using the Automatic Roughness Analyzer (ARAN), frictional resistance measured with the ASTM brake-force trailer, and Pavement Condition Ratings. Overall, the eighteen-year performance of the rehabilitated concrete pavements has been good with acceptable levels of ride quality, frictional resistance and distress propagation.

1.0 Introduction

Until 1989 the traditional method of rehabilitating concrete pavements in Ontario was to remove the distressed concrete full depth or partial depth and replace it with asphaltic concrete. Although these repairs performed well in the short-term, in the long-term the 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 restoration (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. A site was selected in the late

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1980's to demonstrate various restoration techniques, including methods that had not previously been utilized in Ontario.

At the time of construction, this project utilized the most comprehensive CPR techniques undertaken in Canada. The demonstration site is located on a 5 km urban arterial facility in London, Ontario, Highway 126. The highway has since been assumed by the City of London and is now known as Highbury Avenue. Rehabilitation techniques included partial and full depth repairs and diamond grinding on the northbound lanes (NBLs) and an unbonded overlay on the southbound lanes (SBLs). Construction was completed late in the summer of 1989.

2.0 Background

2.1 General. 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%.

The original pavement was constructed in 1963 with 230 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 at the transverse joints. The joints were sealed with preformed neoprene seals.

The pavement serviceability in 1989 varied dramatically between the northbound and southbound lanes. The general pavement condition of the northbound lanes was considered better than average considering the 26-year service life. The distresses on these 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/faulting was exhibited at the joints. The joint seals were approximately 85% intact, although there was significant bond failure between concrete and sealant throughout.

The pavement on the southbound lanes was in much poorer 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 variance in the 26-year pavement performance was attributed to the different aggregate types used in the concrete on the opposing lanes. The quarried aggregate used on the SBLs 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 NBLs, which did not exhibit this D-cracking performance characteristic. 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 overlays
- Full depth repairs and spall repairs with concrete plus diamond grinding on the NBLs; and placement of an unbonded concrete overlay on the SBLs

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 concrete repairs with PCC, diamond grinding and an unbonded concrete overlay was selected, as it best addressed the actual performance deficiencies exhibited by the pavement and provided the longest anticipated service life.

2.2 Design. The PCC rehabilitation techniques utilized on the northbound lanes incorporated the latest designs and construction procedures. The design of the full depth repairs included a minimum repair width of 2.0 m with epoxy coated dowel bars epoxied into drilled holes spaced at 300 mm on both transverse faces. Also, the existing concrete had to be removed in such a manner as not to disturb the underlying granular base.

The partial depth repair requirements included 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 a ministry published report (Kazmierowski et al. 1991).

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 it was overlain by a 180 mm thick unreinforced plain jointed dowelled concrete pavement. Slipforming of the concrete overlay used revised specifications based on experience gained on an earlier Ministry of Transportation Ontario (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.

2.3 Construction. 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).

2.4 Maintenance. Routine maintenance is required to prolong the pavement structure. Maintenance activities included resealing of cracks and joints, as well as some full depth and partial depth concrete repairs performed in NBLs in years 1995 and 2000. SBLs activities included resealing of cracks and joints plus minor full depth and partial depth concrete repairs in 1997.

The remedial work carried out on both northbound and southbound lanes after the last major rehabilitation in 1989 were summarized in Table 1 below:

Table 1. Summary of Remedial Works Performed after Year 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

3.0 Pavement Performance

3.1 Pavement Distress Survey. The visible distresses in the pavement surface were surveyed using the ministry's Rigid Pavement Evaluation Procedure (Chong et al. 1995) to provide an overall indication of pavement performance.

3.1.1 Northbound Lanes – At 3 years of service. 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 a few 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 generally 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 replacement 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 confirm the importance of critical assessment of where partial depth repairs should be used.

3.1.2 Northbound Lanes – At 5 years of service. In 1994 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, requiring full depth repairs, may occur. Consideration should be given to carrying out full depth repairs 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 meter 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 a the time of original construction.

In the summer of 1994, the City of London completed 96 full depth and 16 partial depth repairs along with limited joint and crack resealing. The repairs were completed in the same manner as the original repairs as detailed in (Kazmierowski et al. 1991), except that the surface of the repairs was not diamond ground but tined longitudinally by hand, while the concrete was plastic.

3.1.3 Northbound Lanes – At 10 years of service. In 1999, the survey of the NBLs 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 was also some slight spalling of the repairs and original concrete at the joints and cracks.

3.1.4 Northbound Lanes – At 15 years of service. The NBLs survey in Fall 2004 indicates the pavement is in fair to good condition, with a PCR of 63 in the driving lane, and 70 in the passing lane. MTO's Pavement Condition Rating (PCR) is a composite pavement serviceability index ranging from 0 to 100; with the higher rating indicating a better pavement condition. Routing and sealing of cracks and joints, plus additional full depth and partial depth concrete repairs was carried out in 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 (Kazmierowski, T.J. and Chan, S. 2005).

3.1.5 Northbound Lanes – At 18 years of service. A recent windshield pavement visual condition inspection was carried out in January 2008, after eighteen years of rehabilitation, the NBLs pavement was considered to be in fair condition with an average PCR of 55. There were a few moderate longitudinal and meandering cracks; frequent severe to very severe transverse cracking; moderate faulting throughout; intermittent moderate joint and crack spalling. See Photo 1 below showing a full depth repair exhibiting a slight longitudinal crack. Note the concrete pavement outside the full depth repair is polished, particularly in the wheelpath.


Photo 1. Northbound Lane Full Depth Repair Section

3.1.6 Southbound Lanes – At 10 years of service. There were minimal distresses identified in SBLs for the first 9 years, until 1999 the only distress observed in the SBLs is 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).

3.1.7 Southbound Lanes – At 15 years of service. The SBLs survey in Fall 2004 indicates the pavement is in good condition with a PCR of 70 in the driving lane, and 78 in the passing lane. 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 of moderate severity. In addition, a few severe reflection diagonal cracking and frequent moderate transverse cracks were identified (Kazmierowski, T.J. and Chan, S. 2005).

3.1.8 Southbound Lanes – At 18 years of service. A recent windshield pavement visual condition inspection was carried out in January 2008, after eighteen years of rehabilitation, the southbound lanes pavement was considered to be in fair condition with an average PCR of 52. There were a few to intermittent crescent cracks; intermittent severe longitudinal cracks; moderate longitudinal and meandering cracks; frequent slight to moderate transverse cracking; moderate faulting throughout; few moderate joint and crack spalls; few areas of moderate raveling/coarse aggregate loss. See Photo 2 below for the typical southbound driving lane condition showing a moderate transverse crack. Note that the surface still has a tined macro-texture.

Photo 2. Typical Transverse Crack in Unbonded Concrete Overlay Section

3.2 Smoothness Measurements. Smoothness was measured immediately after construction, prior to opening to traffic, using the California Profilograph. 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 the ministry published report M-76 (Chojnacki 1985). The rougher the pavement, the higher profile index (PI). The average PI 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.

A recent concrete pavement smoothness survey was carried out in December 2007, eighteen years after rehabilitation, using a California Profilograph. The survey was done for both southbound and northbound directions in each lane. The average PI readings for each lane and the number of scallops/bumps are shown in Table 2.

Table 2 Summary of Smoothness Survey in December 2007

Direction	Lane	Profile Index (mm/km)	Number of Scallops*
Southbound (SB)	Driving	853	167
	Passing	587	207
Northbound (NB)	Driving	541	76
	Passing	350	31

* Using 5.0 mm blanking band.

According to the smoothness survey in Table 2, the pavement in the NBLs is significantly smoother than in the SBLs. The PI values and the number of scallops are much lower in the NBLs. Of particular interest is the comparison of initial PI and year-18 PI for the SBLs, 175 mm/km and 169 mm/km versus 697 mm/km and 469 mm/km respectively, which clearly indicates the deterioration in ride quality over the 18 years.

3.3 Roughness Measurements. 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 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 m/km (or mm/m). The IRI scale starts from 0 m/km 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.

3.3.1 Roughness Measurement Northbound Lanes. Roughness in the NBLs improved significantly after the concrete pavement surface was diamond ground in 1989, with an improvement in IRI from initially 3.9 to 0.9 and 2.6 to 0.9 in the driving and passing lanes respectively, see Figure 1. 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. The ride quality levels (smooth, comfortable, uncomfortable) indicated on the graph are converted based on the established ride comfort index (RCI). RCI between 8 to 10 is smooth (IRI = 1.60 to 0.76); 6 to 8 is comfortable (IRI = 3.36 to 1.60); and below 6 (IRI > 3.36) is considered to be uncomfortable ride (Chong et al. 1995).

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 ARAN surveys from 2001 to 2004 indicates the roughness remained constant over these years, with an average IRI of 2.3 and 1.8 for the driving and passing lane respectively. From 2005 to 2007, the driving lane roughness data shows a very mild trend of deterioration with an average IRI of 2.6. The roughness data for the passing lane are still fairly constant between 2005 to 2007, with an average IRI of 1.9.

Overall, the full CPR techniques used on the NBLs are still performing well within the comfortable category after 18 years of service.

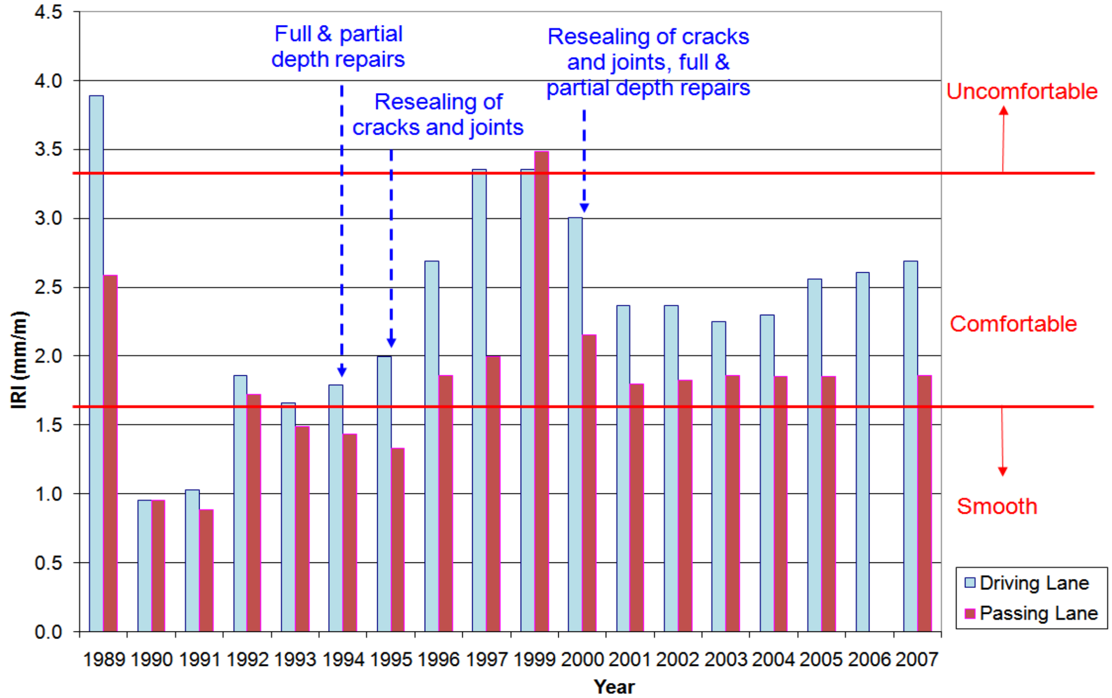

Figure 1. Summary of Roughness (IRI) in Northbound Lanes

3.3.2 Roughness Measurement Southbound Lanes. The unbonded overlay rehabilitation on the SBLs in 1989 resulted in an improvement in roughness (IRI) from 3.9 to 1.7 and 2.8 to 1.6 in the driving and passing lanes respectively; see Figure 2.

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 1.8 and 1.7 in the driving and passing lane respectively.

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

The surveys from 2001 to 2004 indicate the roughness has slowly deteriorated over these years. From year 2005 to 2007, the roughness starts to deteriorate at a faster rate; with the passing lane still within the comfortable range, and the driving lane just bordering into the uncomfortable range.

Overall, the unbonded concrete overlay on the southbound lanes is performing at an acceptable level of ride quality after 18 years of service, and it is well within the anticipated life cycle parameters.

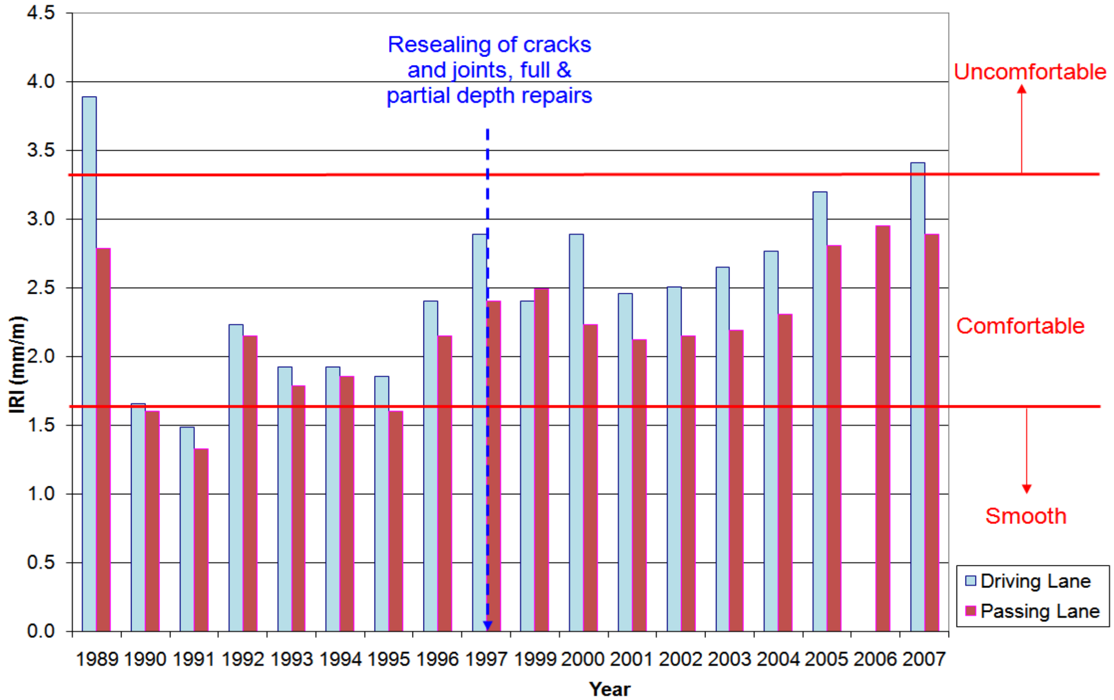

Figure 2. Summary of Roughness (IRI) in Southbound Lanes

3.4 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).

3.4.1. Skid Testing on Northbound Lanes. For the diamond ground surface on the NBLs, the skid numbers improved immediately after construction in 1989, from an average of 25 to 33 and 35 to 38 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).

As shown in Figure 3, the skid resistance has stayed fairly constant since 1990 to 2000, with the driving lane ranging from 24 to 33 (average of 28) and the passing lane ranging from 30 to 39 (average of 35). From 2001 to 2006, the skid resistance starts to decline slowly, from 26 to 21 in the driving lane; and from 32 to 26 in the passing lane.

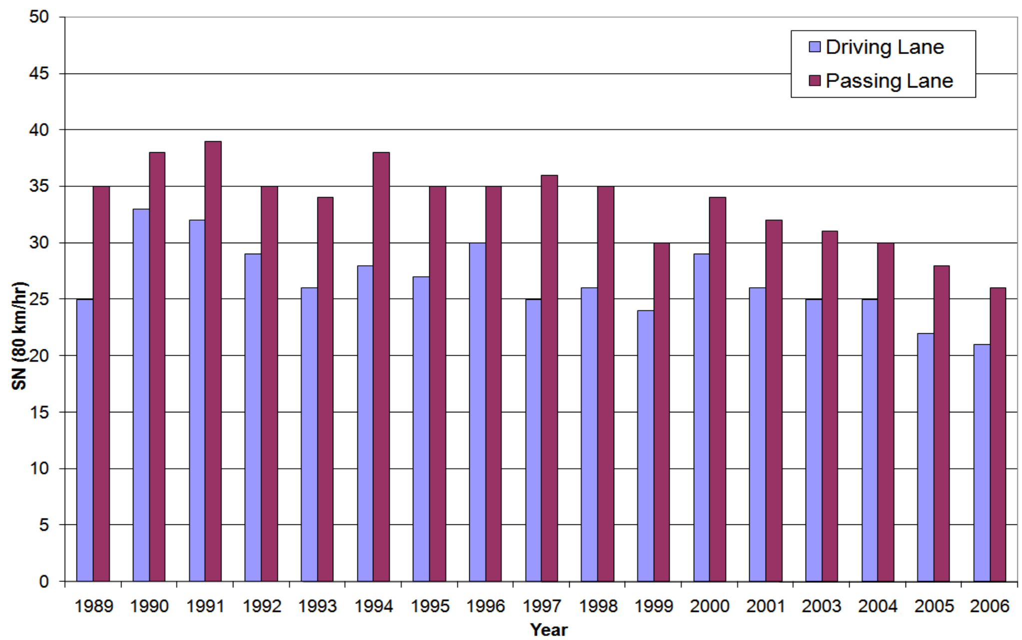

Figure 3. Summary of SN_{80} in Northbound Lanes

3.4.2. Skid Testing on Southbound Lanes. The skid numbers in the SBLs increased from SN_{80} of 18 to 36 (driving lane) and 29 to 40 (passing lane) immediately after the overlay in 1989. See Figure 4 for an annual summary of the skid resistance in the SBLs.

The skid resistance has stayed fairly constant since 1990 to 2000, with the driving lane ranging from 34 to 39 (average of 37) and the passing lane ranging from 39 to 45 (average of 43). From 2001 to 2006, the skid resistance starts to decline slowly, from 33 to 27 in the driving lanes and from 40 to 33 in the passing lane.

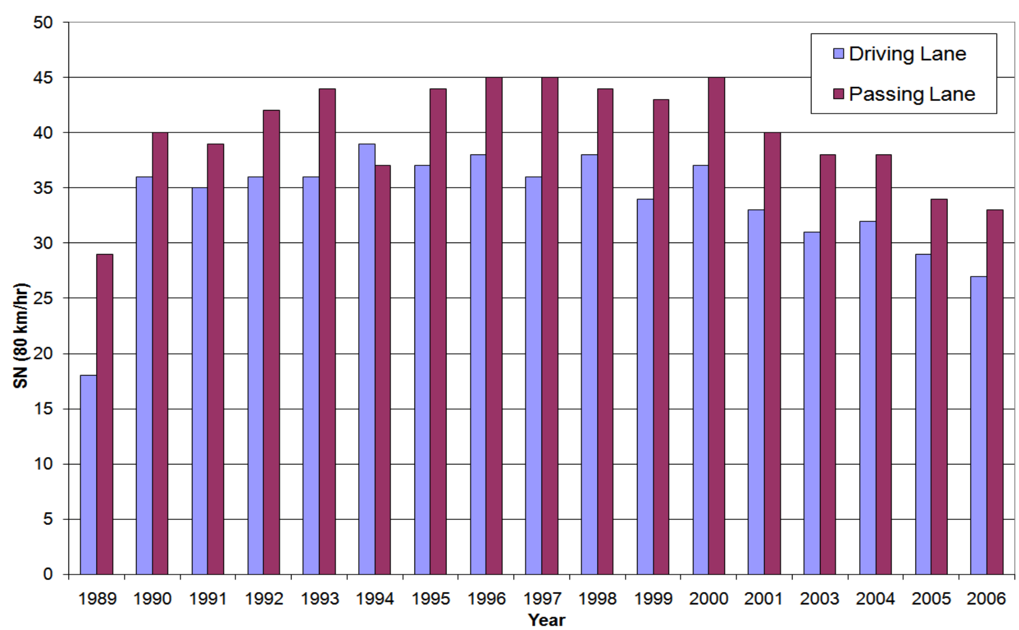

Figure 4. Summary of SN_{80} in Southbound Lanes

3.4.3. Skid Testing Comparison on Northbound and Southbound Lanes. The average skid numbers for NBLs from 1990 to 2006 are 27 for driving lane and 34 for passing lane. The average skid numbers for SBLs from 1990 to 2006 are 35 for driving lane and 41 for passing lane. These average skid numbers indicated NBLs with diamond ground texture are approximately 7 to 8 units lower than the SBLs with a burlap drag and tine texture. For the diamond grinding, 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 tine texture in the southbound lanes, resulting in an earlier reduction in measured frictional properties.

3.5 Sound Level Measurements. Sound level measurements were undertaken prior to and following 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 results indicated that the diamond ground surface was approximately 1 dBA quieter than the old worn concrete surface and the MTO sample. The newly diamond ground surface (1.5 mm deep, 5 mm spacing) was only 1 dBA louder than a newly open graded asphalt friction course, which is used for its low noise characteristics and good skid resistance. The MTO sample obtained from the ministry published report (Jung et al. 1985) was derived by testing several hundred vehicles on a variety of pavement surfaces, predominantly asphaltic concrete, across the province.

The burlap dragged and transversely tined concrete surface (tine dimension of 3 mm wide, 4 mm deep, and 16 mm tine spacing) was approximately 1 dBA noisier than the old worn concrete and MTO sample. For further details on the sound emission measurements see the published TRB Record (Kazmierowski et al. 1991).

4.0 Conclusions & Recommendations

Highway 126 pavement, constructed in 1963, has provided commendable service for almost 45 years, and is now reaching the end of its 1st CPR service life. The major reason of its good performance was timely maintenance and rehabilitation.

Monitoring the serviceability of the pavement over the last eighteen years has indicated the concrete pavement rehabilitation techniques used have exhibited acceptable performance on both the northbound (diamond grinding, full and partial depth repairs) and southbound (180 mm unbonded concrete overlay) lanes. In general, the northbound lanes are performing better than the southbound lanes; except the skid resistance in NBLs (diamond ground) has deteriorated at a faster rate than SBLs (tined texture). Table 3 below is a summary indicating the performance of the pavement after eighteen years of service:

Table 3. Summary of Pavement Performance after 18 Years of Rehabilitation

PERFORMANCE AT YEAR 18	SOUTHBOUND (SBL)	NORTHBOUND (NBL)
Distresses	• Few to intermittent crescent cracks • Intermittent severe longitudinal cracks • Moderate longitudinal and meandering cracks • Frequent slight to moderate transverse cracking • Moderate faulting throughout • Few moderate joint and crack spalls • Few areas of moderate raveling/coarse aggregate loss	• Few moderate longitudinal and meandering cracks • Frequent severe to very severe transverse cracking • Moderate faulting throughout • Intermittent moderate joint and crack spalling
PCR*	52	55
PI*	720 mm/km	446 mm/km
No. of Scallops/Bumps	374	107
IRI*	3.2 m/km	2.3 m/km
Skid Number *(at year 17)**	30	24
Ride Quality	Comfortable (with localized uncomfortable areas)	Comfortable (with few uncomfortable areas)

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

Based on the traffic data in year 2000, the SBLs truck traffic is 40% higher than the NBLs and that explains why the smoothness (PI) and roughness (IRI) values for the SBLs is significantly higher than the NBLs (Table 3).

The NBLs, which were treated with diamond grinding, full depth and partial depth repairs are performing acceptably. The diamond grinding significantly restored ride quality of the old concrete pavement for the eighteen-year monitoring period. Initially, diamond grinding at the NBLs 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 18 years of service, measured skid resistance of the diamond ground surface was approximately 7 to 8 skid units lower than that of the burlap drag/tined surface on the SBLs.

Pavement monitoring in the NBLs 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 NBLs, in conjunction with core analysis, confirm 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.

The current major issues for both northbound and southbound lanes are the surface texture with relatively low skid numbers; moderate faulting of joint and cracks; and the presence of severe cracking and spalling at isolated areas. A possible 2nd generation CPR strategy is to treat this pavement with localized full depth concrete repairs and diamond grinding. The pavement still appears to be generally within the limit values for diamond grinding (average faulting of less than 12 mm and IRI less than 3.0 m/km) for this road. Diamond grinding should eliminate faulting at joints and cracks in both directions and remove construction curling and warping of the slabs in the SBLs; thus providing a smooth riding surface. It will also enhance the surface texture and friction characteristics and reduce tire/pavement noise emissions. The severe cracks and spalls should be repaired using either full depth cast-in-place or precast slab repairs. A detailed concrete pavement rehabilitation design including a full geotechnical investigation is required to finalize the rehabilitation strategy.

Monitoring the long-term pavement performance, associated with each type of concrete pavement restoration (CPR) technique, has been crucial in order to model CPR performance and obtain data on life cycle costing of these CPR strategies. The serviceability of this concrete pavement after 18 years since major rehabilitation has been fair to good, with acceptable levels of ride quality and frictional resistance.

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.

5.0 Acknowledgement

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