# PERFORMANCE OF ONTARIO'S FIRST MAJOR REHABILITATION AND OVERLAY PROJECT

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## ABSTRACT

The rehabilitation of concrete freeways has challenged highway authorities for many years. This challenge has been met with the utilization of various techniques such as full and partial depth repairs, diamond grinding and unbonded overlays. In 1989, the Ontario Ministry of Transportation constructed a project to demonstrate the feasibility of these techniques in their jurisdiction, and to monitor the long term performance of the various applications.

The project incorporated the use of full depth and partial depth repairs followed by diamond grinding and joint sealant replacement on the northbound lanes (NBL). The southbound lanes (SBL) received an 180 mm thick plain jointed unbonded PCC (JPCP) overlay.

The rehabilitation project has now been in service for three years and a number of performance characteristics have been monitored. These include pavement roughness and skid resistance with testing completed on an annual basis while pavement load response, condition surveys and noise emissions testing have been undertaken on an as need basis. The results of this testing and analysis are presented in this paper.

## INTRODUCTION

The rehabilitation of concrete pavements has undergone significant development and refinement recently. These advancements have provided engineers with the opportunity to restore concrete pavements with concrete, thereby maintaining the integrity of the pavement and also retaining the benefits of concrete. With the evolution of concrete rehabilitation techniques, the Ministry of Transportation reviewed their potential for use on a highway project in Ontario. A site was selected in the late 1980's to demonstrate a number of restoration techniques previously used, as well as methods that had not been utilized in Ontario. In addition to providing an opportunity to assess the feasibility of constructing the diverse rehabilitation techniques, a program of long-term monitoring was also planned and initiated.

This project utilized the most comprehensive rehabilitation techniques undertaken in Canada. The demonstration site is located on a 5 km long roadway, Highbury Avenue (previously Highway 126) in London, Ontario. The project included PCC pavement rehabilitation techniques ranging from partial and full depth repairs to diamond grinding on the northbound lanes and an unbonded overlay on the southbound lanes. Construction was completed in the late summer of 1989, and monitoring commenced that fall.

## BACKGROUND

### General

Highbury Avenue, formerly Highway 126 prior to being assumed by the City of London, is located approximately 200 kilometres west of Toronto. The road is a four-lane divided arterial, with traffic volumes ranging from 19,000 to 30,000 AADT, of which 3,000 are commercial vehicles.

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

The pavement serviceability varied dramatically between the NBL and SBL. The pavement condition on the northbound lanes was relatively good considering the 26 year service life. The main distresses were: approximately three transverse cracks per slab with one being a severe working crack and two slight cracks, some spalling at joints and cracks, and a moderately polished surface. Typically the transverse contraction joints had seized which lead to the working severe cracks. The joints exhibited minimal stepping and good load transfer. The joint seals were 80% to 90% intact, although significant bond failure between concrete and sealant was apparent throughout.

The pavement condition on the southbound lanes was more distressed, with full and partial width transverse cracks in each slab, severe D-cracking, spalling of all joints throughout and moderate pavement edge distress. Deterioration had advanced to the stage where routine maintenance activities could no longer preserve a safe riding surface.

The variance in pavement performance was attributed to the two types of aggregate used in the concrete on the opposing lanes. The quarried aggregate used on the SBL 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 NBL.

Three rehabilitation strategies were considered for use on this project and included: crack and seat with an asphalt overlay; repair of the PCC pavement with hot mix and a multiple lift asphalt overlay; and repair with concrete and diamond grinding on the NBL and placement of an unbonded concrete overlay on the SBL.

The initial construction costs for the three options varied from 1.5 to 1.8 million dollars, however, life cycle cost analysis which included anticipated service life indicated that the costs 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. This strategy also provided the longest anticipated service life.

### Design

The PCC rehabilitation techniques utilized on the NBL incorporated current design philosophies and procedures. The design of the full depth repairs included a minimum repair width of 2.0 m and epoxy coated dowel bars epoxied into drilled holes spaced at 300 mm on both transverse faces. Partial depth repair requirements included perimeter sawcutting to a 50mm depth, 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 reference [1].

The design of the unbonded concrete overlay on the SBL included the placement of a 20 mm asphaltic sand hot mix bondbreaker over the existing pavement surface. This was then overlain by a 180 mm thick unreinforced plain jointed concrete pavement (JPCP). Slipforming of the concrete overlay used revised specifications based on experience gained on the Highway 3N experimental pavements [2, 3]. Skewed randomized joints were utilized with no effort made to avoid matching the underlying joints.

### Construction

Generally, construction of the concrete repairs, diamond grinding and the unbonded overlay went smoothly following the initial start-up difficulties. Details of the construction procedure and material testing have been published in an earlier report [1].

## PAVEMENT PERFORMANCE

### Roughness Measurements

The ride of a pavement is the most visible sign of pavement condition to the travelling public, therefore, the collection and analysis of the ride condition or roughness data was undertaken. Roughness was measured, just after construction and prior to opening to traffic, and on a yearly basis since construction. The California Profilograph was used after construction and the Portable Universal Roughness Device (PURD) for ongoing monitoring.

The profilograph measurements were taken on the new concrete overlay with readings taken in both wheelpaths of the driving and passing lanes. For details on the function and operation of a profilograph see Reference [4]. The average profilograph reading, profile index, for the length of the project in the driving lane was 11.1 in/mile and 10.7 in/mile in the passing lane. These readings were taken prior to diamond grinding 21 locations where the roadway did not meet the surface tolerance specifications of 3 mm in 3 m. These values compare reasonably well with the numbers obtained by other authorities constructing considerably larger quantities of concrete pavement. Many of these agencies have performance specifications requiring a ride ranging from 12 in/mile to as low as 7 in/mile with incentive bonuses for smoother pavements.

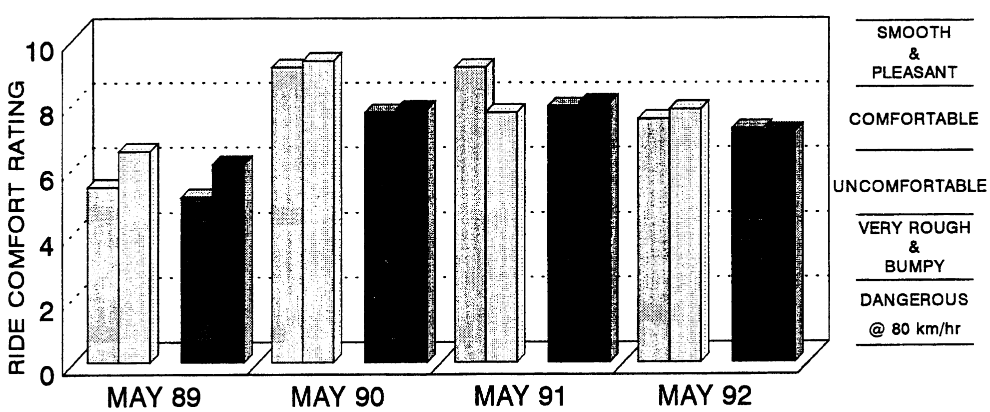
Roughness measurements using the PURD were also taken. The PURD is a trailer mounted accelerometer-based mechanical measuring device operated at constant highway speed, that uses the root mean square vertical acceleration, in milli g's, of the trailer axle to measure roughness. PURD measurements are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

Testing for roughness was undertaken using the PURD prior to rehabilitation, and after rehabilitation just prior to opening to traffic and on an annual basis since. All four traffic lanes were tested, the northbound driving and passing lanes (NDL and NPL) and southbound lanes (SDL and SPL).

Generally ride was improved significantly on the NBL (diamond ground surface) with an improvement in ride comfort rating (RCR) from 5.4 and 6.5 to 9.1 and 9.3 in the driving and passing lanes respectively. The ride data for the diamond ground surface indicates a very smooth surface is obtained with a somewhat rougher surface for the concrete overlay which is only slightly rougher than the ride achieved on a new asphalt surface.

The unbonded overlay on the SBL exhibited an improvement in ride from 5.1 and 6.1 to 7.7 and 7.8 in the driving and passing lanes respectively, see Figure 1. The newly tined surface texture on the unbonded overlay was very aggressive and may have contributed to some of the initial roughness. The wearing of this aggressive texture may also have been accountable for the very slight improvement in ride between post construction readings of 7.7 and 7.8 compared to the May 91 readings of 7.9 and 8.0 in the driving and passing lanes respectively.

SUMMARY OF PURD ROUGHNESS SURVEY




| | | | | |
|---|---|---|---|---|
| NDL | 5.4 | 9.1 | 9.1 | 7.5 |
| NPL | 6.5 | 9.3 | 7.7 | 7.8 |
| SDL | 5.1 | 7.7 | 7.9 | 7.2 |
| SPL | 6.1 | 7.8 | 8 | 7.1 |

FIGURE 1

The more recent roughness measurements show a moderate decline in RCR for both the NB & SBL's. The northbound lanes have shown the most significant declines in RCR of 15 to 17 percent, from 9.2 and 9.1 to 7.8 and 7.5 in the driving and passing lanes respectively. The southbound lanes have shown an 8 to 10 percent decline in RCR from 8.0 and 7.9 to 7.1 and 7.2 in the driving and passing lanes respectively.

This rapid deterioration in RCR over the past year may be attributable to the very unusual environmental conditions in 1992. Precipitation levels over the last year along with a mild wet winter may have been a significant factor in reducing the RCR. Along with these environmental conditions the NBL has also exhibited additional distress in the last year which is detailed in the following section. PURD values for 1992 were retested with excellent correlation to confirm accuracy.

**Pavement Distress Survey**

The visible distresses in the pavement surface were surveyed to provide an overall indication of performance. Generally the pavement on the NBL is in good condition and the SBL are in excellent condition.

The only distress observed in the southbound lanes is very slight longitudinal cracking in eleven driving lane slabs. This can probably be attributed to inadequate longitudinal joint construction as the cracks were visible soon after construction. No transverse or reflection cracking was observed, which correlates with other agencies' experience [5].

Since rehabilitation of the northbound lanes there has been some occurrence of additional distress. Most of the highly visible distress is related to the deterioration of partial depth repairs, however there are a number of other minor distresses. Some of the less visible distresses include the progression of hairline cracks to very slight cracks (<3mm wide). This is probably due to stress relief during construction of full depth repairs. A few of the cracks which were originally slight ( 3-12mm ) prior to crack sealing in 1989 have begun to fault with an average measurement being 4 mm and a range of 1 to 9 mm. There are also a few cracks and joints which are exhibiting some spalling and corner cracking.

Performance of full depth repairs on the NBL has been very good overall. Only 7 of 266 repairs are exhibiting distress which is typified by very slight longitudinal cracking in the right wheelpath of the driving lane. In a few locations these cracks are beginning to propagate into the adjacent slabs. No signs of movement or faulting of the repairs was observed.

A significant reduction in the performance of the partial depth repairs was observed when compared to the full depth repairs. Approximately half of the 75 partial depth repairs are exhibiting distress. The majority of the failures are in the driving lane and in the outside one third of the slab. This is likely due to higher stress levels at the outside of the pavement as there are no integral shoulders. A few of the partial depth repairs at joints are exhibiting severe distress along the full width of the driving lane.

A limited investigation of a few partial depth repairs was undertaken. The investigation revealed that where the partial depth repair extended to the level of the dowel bars, deterioration occurred rapidly. This reaffirms the generally accepted practice of not undertaking partial depth repairs where spalling may be occurring at both the top and bottom of the slab [6]. This was also confirmed by cores taken through some slabs which did not exhibit any cracking. In these uncracked repairs the replacement did not exceed 1/3 the slab depth and bond between the old and new concrete was excellent. Bond between the old and new concrete was also very good in the deteriorated repairs indicating the weakness or spalling in the slab extended below the repair level. These findings reaffirm the importance of critical assessment of where partial depth repairs are used.

**Load Response Characteristics**

In order to evaluate and compare the load/deflection characteristics and the joint load transfer efficiencies of the rehabilitated pavement a non-destructive load testing program was conducted.

Prior to rehabilitation slabs were randomly selected for testing of corner to centre slab deflection ratios and load transfer efficiency across joints. This information was used for design to determine the presence of voids beneath the slabs and to establish the need for stabilization of the joints by subsealing.

Deflection testing was completed using the Dynatest 8000 falling weight deflectometer (FWD). The majority of testing was performed early in the day to minimize the influence of slab curl resulting from differential slab temperatures.

Deflection testing undertaken prior to rehabilitation indicated the pavement in both directions was in a stable condition. Load transfer efficiencies across joints or cracks varied from a mean value of 78 to 89 percent on the NBL and from 67 to 83 percent on the SBL. Load transfer efficiency of 70 to 100 percent is generally observed in new pavements and is considered good [6]. These values also indicated that voids beneath the slabs were very small or non-existent. The corner to centre deflection ratio testing on the old pavement verified acceptable slab support and the absence of voids based on mean values ranging from 3.3 to 5.9 in the NBL and from 4.4 to 4.8 in the SBL [1].

Following rehabilitation, deflection testing was undertaken at the same locations or at the nearest point to the preconstruction testing. Testing was again undertaken in the summer of 1992 to evaluate the performance of the repairs and to provide a general overview of the rehabilitation performance.

Testing on the unbonded overlay, indicated an improvement in both load transfer and corner/centre deflection ratios as would be expected [1]. The post construction load transfer testing indicated mean values ranging from 88 to 91 percent, an increase of 5 to 24 percent over mean preconstruction levels. The load transfer was also more uniform among the joints tested as evidenced by the reduction in the range of test values.

The three year test data shows a slight improvement in load transfer efficiencies to mean values of 93 and 96 percent with a significant reduction in the range of the transfer efficiencies. See Figure 2. The significant reduction in range may be attributed to transfer of all joints into a working state as compared to the early life of the pavement where a limited number of joints are operational. The subsequent redistribution of working joints would lead to a more uniform performance.

**Load Transfer Results**

Southbound Driving Lane

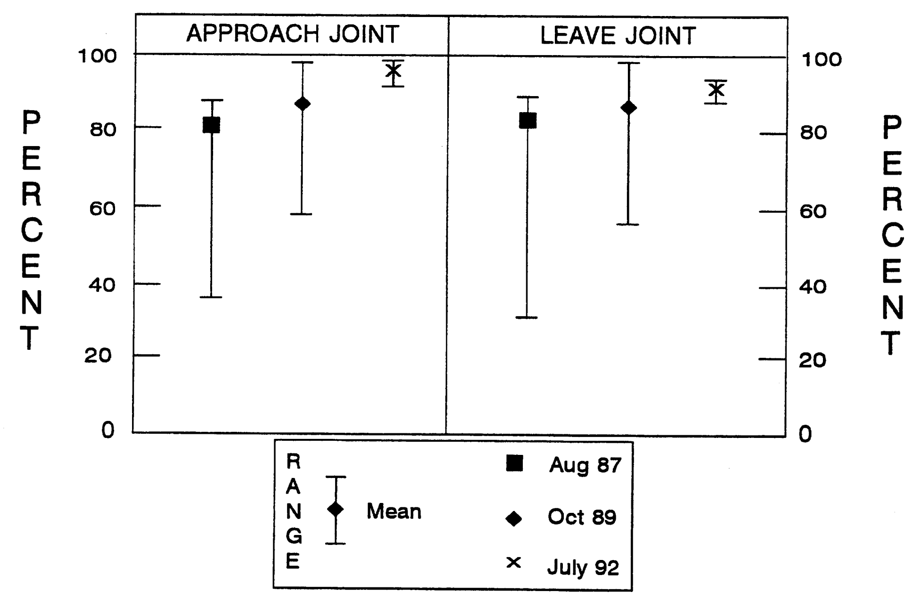



FIGURE 2

On the NBL, post construction and three year testing was carried out on unrepaired joints and cracks as well as repaired sections. All results for the unrepaired sections in the passing lanes were very similar to previous data with some minor deviations due to normal test variations, refer to Figure 3. There was however a significant reduction in the 3 year tests for the driving lane. Load transfer efficiencies dropped from an approximate average of 80% to 50% in the driving lane. This is probably due to the high percentage of truck traffic using this lane which results in a reduction of aggregate interlock at cracks and joints. This hypothesis is substantiated by the observation of faulting at a few cracks.

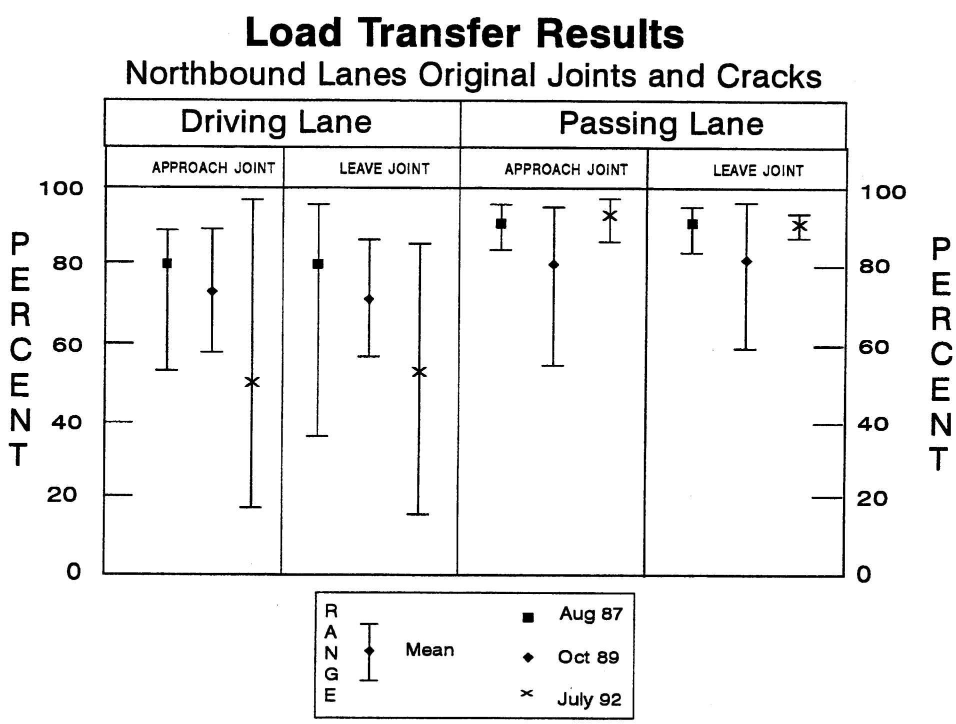



FIGURE 3

The initial results of FWD testing across the full depth repair joints indicated load transfer had been reduced in areas where improvements were anticipated. A general pattern of lower load transfer values, in the order of 20 percent, was observed when loads were applied on the repair itself versus loading on the old concrete. This data is shown graphically on Figure 4. Subsequent testing on other projects revealed that the combination of surficial loosening/softening of the granular base in the repair area and the use of grease as debonding agent on dowel bars contributed to this situation. After three years, FWD test results indicate this anomaly is no longer present.

Load transfer across the repair joint has apparently stabilized. This is probably due to a number of factors. In the driving lanes the load transfer values have generally declined to equitable levels in the 60 percent range, probably due to slight deterioration of the dowel support. In the passing lanes the load transfer efficiencies have improved significantly. This may be due to a general stabilization of the full depth repair and adjacent slabs. Generally there is a very wide range in performance, especially in driving lanes.

Overall performance appears to be satisfactory with slightly lower efficiencies in the driving lanes than would be anticipated.

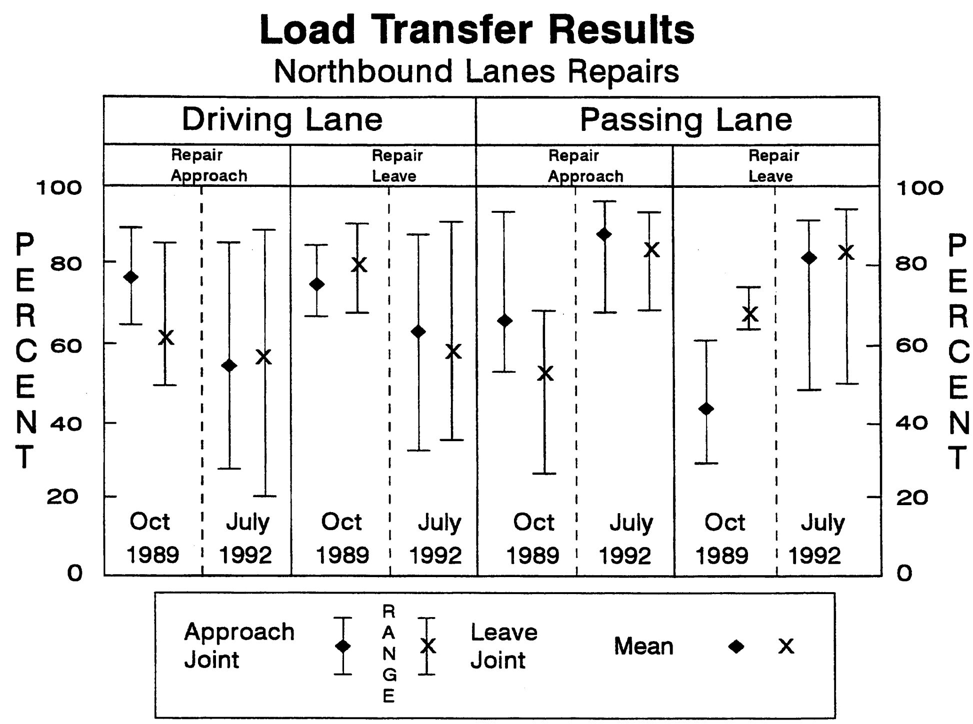
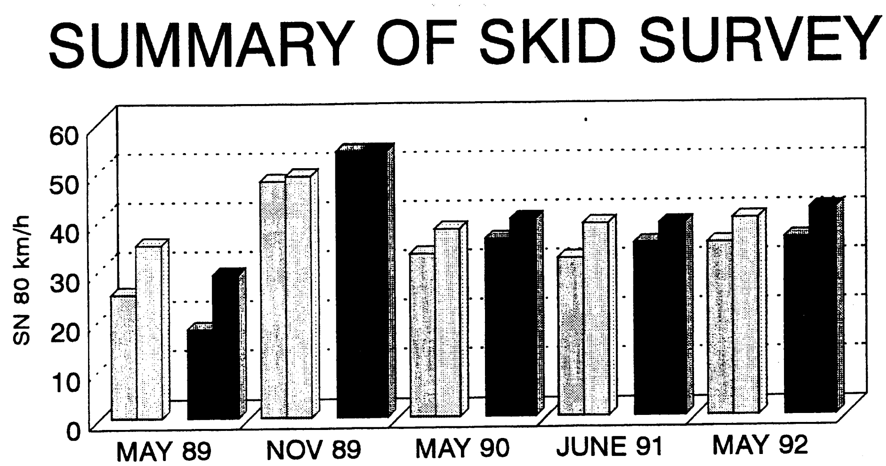



FIGURE 4

**Skid Testing**

In North America the most commonly accepted method of measuring skid resistance is the skid 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.

Results of skid resistance testing are detailed on Figure 5. The test results indicate a substantial increase in skid resistance on both the northbound and southbound lanes immediately following rehabilitation as one would expect. The skid numbers in the southbound lanes increased to an average SN 80 of 55 for both lanes from an SN 80 of 18.3 and 28.9 in the driving (SDL) and passing lanes (SPL) respectively. On the diamond ground surface the skid numbers improved from an average of 24.8 and 35.1 to 48.0 and 50.4 for the driving (NDL) and passing lanes (NPL) respectively.




| | | | | | |
|---|---|---|---|---|---|
| NDL | 25 | 48 | 33 | 32 | 35 |
| NPL | 35 | 49 | 38 | 39 | 40 |
| SDL | 18 | 54 | 36 | 35 | 36 |
| SPL | 29 | 54 | 40 | 39 | 42 |

FIGURE 5

Skid numbers of this magnitude indicate a good friction factor and are similar to numbers obtained on premium hot mix surface courses [7]. The corduroy type of texture developed by grinding visually indicates high frictional resistance. The ridges improve the surface macro-texture and provide an escape route for moisture under a tire. The broken ridge surface, concrete fractured between the saw blades, improves micro-texture.

As anticipated in the first year the initial sharp projections of both the ground and drag/tined surfaces have been worn, polished or sheared off by traffic and snow removal operations. This elimination of sharp projections reduced SN 80 values from values in the high 40's and mid 50's to the mid 30's for the driving lanes and low 40's for the passing lanes. Based on previous experience this trend was anticipated, however it is interesting to note that the diamond ground surface is performing similarly to the tined surface at present.

**Sound Level Measurements**

Sound level measurements were undertaken prior to and following rehabilitation to provide data for future projects. Highway noise is becoming an ever increasing environmental concern to the public, therefore it is important to collect and maintain this information.

The procedure used to measure pavement noise on this project was to measure noise from individual vehicle passbys at a 15 m distance from the travelled lane. The results are plotted on Figure 6 and indicate that the diamond ground surface was approximately 1 dBA quieter than the old worn concrete and the Ministry of Transportation of Ontario (MTO) sample. The ground surface was only 1 dBA louder than an open graded asphalt friction course which is used for its low noise characteristics and good skid resistance. The MTO sample is obtained from Reference [8] and 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 was approximately 1 dBA noisier than the old worn concrete and MTO sample. The new concrete on the SBL may become quieter as some of the very coarse tining wears off.

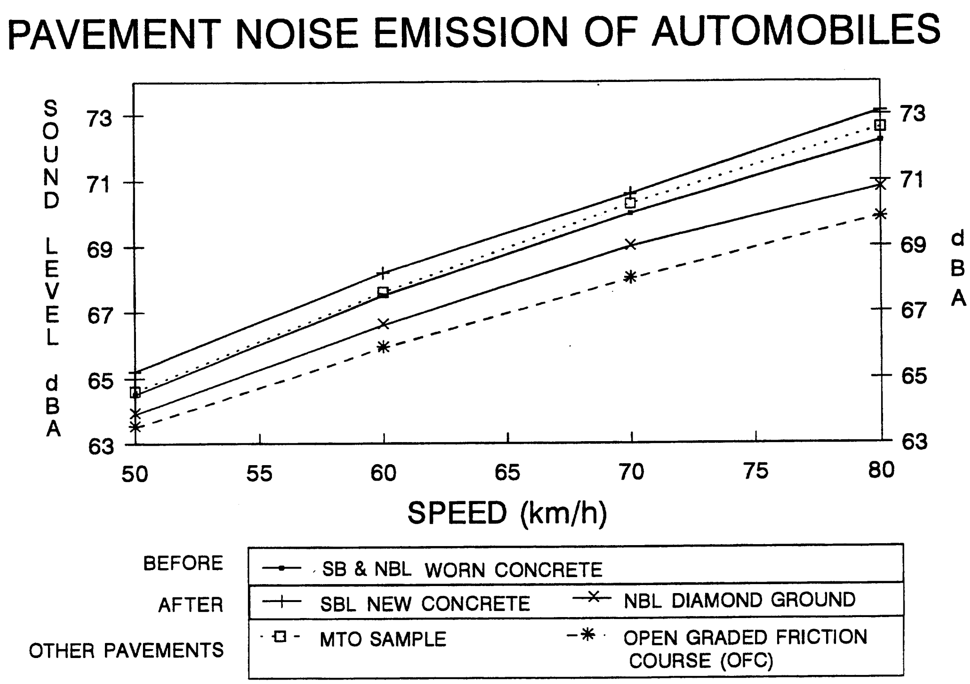



FIGURE 6

**CONCLUSIONS**

1. Significant improvement in ride is attainable using diamond grinding techniques. Post construction measurements indicate values as good as those attained on a premium hot mix surface. Ride or roughness will continue to be monitored.

2. Visual observation in conjunction with core analysis of partial depth repairs confirms the importance of not only observing quality construction procedures but also of critical selection of repair locations.

3. Pavement monitoring has revealed that where existing pavement joints are seized and full depth repairs are constructed at mid-slab working cracks future deterioration of secondary minor transverse cracks may occur.

4. Initial discrepancies between load transfer across repair joints was of concern, however, these inequities have stabilized with time.

5. Performance of the unbonded overly has been good to date with no indication of reflection cracking.

6. Skid resistance on both the diamond ground and burlap drag/tined surface has stabilized at relatively high levels.

7. Overall Highway 126 has shown that a major concrete rehabilitation project in Ontario's wet-freeze environment provides a high level of serviceability after 3 years of service.

8. Future Work:

   - improve assessment criteria for selecting partial depth repair locations
   - continued monitoring of load transfer performance and faulting of cracks/joints in the northbound driving lanes
   - investigate and monitor long term skid resistance of diamond ground surfaces

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