TEN YEARS EXPERIENCE WITH EXPERIMENTAL CONCRETE PAVEMENT SECTIONS IN ONTARIO

Thomas J. Kazmierowski, P. Eng.
Alison Bradbury, P. Eng.
Pavement Design, Evaluation and Management Section
Ministry of Transportation of Ontario
Downsview, Ontario

In 1982, four experimental sections of rigid pavement were constructed on Highway 3, southeast of Windsor, to assess the comparative performance and overall serviceability of various innovative concepts in concrete pavement design.

The demonstration project was evaluated and reported on in 1985 and 1988 after three and six years of service, and most recently in 1992 after ten years of service.

The concrete pavement consisted of four plain jointed undowelled concrete pavement (JPCP) sections varying in design and length.

The performance of the pavement sections is described in terms of load transfer and centre slab deflections based on Falling Weight Deflectometer (FWD) testing, roughness, skid resistance, joint movement and a crack survey. Observations of noise levels, traffic volumes, and surface texture are discussed. A brief summary of the design and construction details, plus the performance results of an ongoing ten year monitoring program are documented in this paper. Conclusions based on ten years of performance results indicate the higher levels of serviceability achieved with the free draining base section and full depth section.

INTRODUCTION

In 1982, the Ministry of Transportation of Ontario constructed a 15.6 km length of experimental concrete pavement which incorporated varying thicknesses of plain jointed, undowelled concrete pavement (JPCP) on erodible and non-erodible bases, two shoulder designs, plus two surface textures.

The concrete pavement consisted of four sections varying in design and length:

Section I - 4.8 km of 305 mm thick undowelled JPCP with 76 mm thick hot mix shoulders

Section II - 5.2 km of 203 mm thick undowelled JPCP with 76 mm thick hot mix partially paved shoulders over 102 mm open graded drainage layer (bituminous treated permeable base)

Section III - 0.8 km of 203 mm thick undowelled JPCP with 203 mm tied-in-place concrete shoulders over 127 mm lean concrete base

Section IV - 4.8 km of 178 mm thick undowelled JPCP with 178 mm tied-in-place concrete shoulders over 127 mm lean concrete base

In order to assess comparable skid resistance and noise level characteristics, the pavement surface was textured by dragging either an astroturf or burlap mat behind the paver prior to transversely tining the concrete surface.

The following performance features have been continuously monitored over the ten year life of the experimental pavement: roughness, skid resistance, load/deflection characteristics, pavement condition surveys, cracking, joint movement and traffic volumes.

This paper summarizes these measurements at the completion of construction and over the ten year monitoring period, discusses observations about the performance and serviceability of the various designs, the noise characteristics of the different textured surfaces, and presents conclusions based on these results.

DESIGN PARAMETERS

An overview of concrete pavement design and construction from the Ontario perspective is documented in [1].

The experimental pavement on Hwy. 3N was designed as a 7.3 m wide, two-lane new facility project with 0.6 m partially paved shoulders, located some 24 km southeast of Windsor. Refer to site plan, Figure 1. All pavement tapers, speed change lanes, connecting sideroads, etc., were designed as conventional hot mix pavement. The majority (95%) of the test sections were constructed on fill, typically the parent silty clay indigenous to the area. The topography is flat over most of the alignment with some relief at the south end. Details of the existing soil conditions are contained in [1].

Traffic Conditions

The Average Annual Daily Traffic (AADT) anticipated in 1978 was 4400, with 13% classified as commercial traffic. At the time of design, the 20 year projected AADT was 8600. This represents approximately 3.5 million (80 kN) Equivalent Single Axle Load (ESAL) applications over the 20 year design period.

Traffic readings taken in the spring of 1988 indicate an Average Annual Daily Traffic count of 9450 with a measured commercial traffic count of 10%. This figure indicates a 30% increase in traffic over three years and well beyond the 1998 (20 year) projected AADT of 8600 used in design.

The actual accumulated number of ESAL in 1988 was 1.0 million as compared to the predicted six year accumulated ESAL of 0.7 million used in design. This represents a 40% increase in traffic loading over six years and a 100% increase over the 20 year design life when compared to original projected design traffic loadings.

Alternative Designs

In consideration of these parameters a benchmark design of 203 mm thick plain jointed, undowelled concrete pavement over 127 mm lean concrete base was selected as an alternative to the initially approved 305 mm full depth bituminous design. The alternatives selected represent variations in design strategy utilizing different erodible bases and subbases [2] while maintaining similar strength equivalences. Detailed sections of the four pavement designs are shown on Figures 2, 3, and 4.

In consideration of the anticipated light truck traffic, it was felt that sufficient load transfer between slabs would be achieved through aggregate interlock by closely spaced undowelled joints. To ensure a smoother ride over the joints and improved joint performance a randomized pattern of 4, 5.8, 5.5, 3.7 m for transverse joints skewed at 1:6 was recommended.

Joints were sawn to a width of 13 mm and a depth of 25 mm, blown clean and filled with a hot poured rubberized sealant.

MATERIALS AND CONSTRUCTION

Details of the construction techniques and records of material testing have been published in earlier reports [1,3]. The following abbreviated details are highlighted for this paper.

A 100 mm subdrain wrapped with a knitted filter sock was installed on all sections to the required grade and location by ploughing [4,5] and reinstating the displaced granular material in one continuous operation.

In order to ensure long term performance and minimize 'D' cracking, high quality aggregates with low absorption characteristics were used in all concrete pavement and bases. Aggregate sources whose material met acceptance criteria for freeze/thaw testing (maximum -0.04% expansion at 250 freeze/thaw cycles) as per ASTM C666, Procedure A were specified. The coarse aggregate used on site was a Manitoulin Island Dolomite.

All lean concrete base and concrete pavements were placed by slip-forming techniques.

The specification requirements and results of the quality assurance acceptance testing for the lean concrete base and concrete pavements are detailed in [1].

Drainage Layer

The Open Graded Drainage Layer (OGDL) used in Section II is an open textured, uniformly graded, asphalt cement stabilized aggregate which was placed in a 100 mm lift directly on the clay subgrade using a conventional hot mix paver. This free draining, bituminous treated permeable base course material is designed to expedite internal drainage of the pavement structure while providing sufficient stability to resist densification and shear distortion under construction traffic.

The OGDL was specified as a modified hot mix aggregate with a 90/10 coarse to fine aggregate ratio with 2% asphalt cement binder (85/100 penetration). The coarse aggregate was 100% crushed material. The ratio of coarse to fine was modified during construction to improve stability.

Field permeability testing indicated a lateral flow velocity of 2.8×10^{-1} cm/sec. Based on laboratory permeability testing of cored field samples the average vertical permeability was 1.2×10^{-1} cm/sec. The permeability of the base course was such that water poured on to the surface drained instantaneously into the mat and did not have time to flow over the surface.

PAVEMENT PERFORMANCE

Profilograph Readings

The pavement test sections were profilographed with a California Type Profilograph immediately upon setting of the concrete. Profiles were taken along the four wheel paths in the direction of paving. For description and operation details of the profilograph, refer to [6].

The results indicate that in the sections where two lanes were slipformed integrally (Sections I and II) and controlled by a one-side-stringline grade control, the closest wheelpath to the guide consistently had the lowest Profile Index, while each adjacent wheelpath had a correspondingly higher average Profile Index.

In the sections where each lane plus shoulder were individually slipformed (Sections III and IV), there is no clear distinction between the inner and outer wheelpaths in the first lane placed. However, in the second lane placed, the inner wheelpath is consistently smoother than the outer wheelpath.

These results are indicative of the problems, particularly with multi-lane width paving, associated with a paver controlled by a single grade control, except on the subsequent single lane construction (Section III and IV, E.B.L.) where a travelling ski plus stringline were utilized. For more detailed information on these results see [1].

The final corrected pavement surface was acceptably smooth meeting the allowable requirements for Profile Index.

Pavement Roughness

Two methods have been used to measure pavement roughness over the ten year monitoring period, from 1982 to 1988 measurements were taken with a Mays Ride Meter (MRM) and from 1989 to 1992 with a Portable Universal Roughness Device (PURD).

The MRM is a road response type measuring device which is mounted to the rear axle of a vehicle or trailer operated at a constant speed. The MRM uses the vertical distance the vehicle travels with respect to the rear axle in in/mile to measure the roughness.

The PURD is a trailer-mounted accelerometer-based measuring device operated at a constant speed on the highway. It uses the root mean square of vertical acceleration of the trailer axle (PURD) to measure roughness. These are converted into a Riding Comfort Rating (RCR) as follows [7]:

$$RCR = 26.64 - 7.38 \log_{10} (PURD)$$

In order to compare the results produced by the MRM and the PURD, the results from the MRM were correlated with PURD results and converted to a RCR scale of 0 to 10. A summary of the roughness survey is shown on Charts 1 and 2.

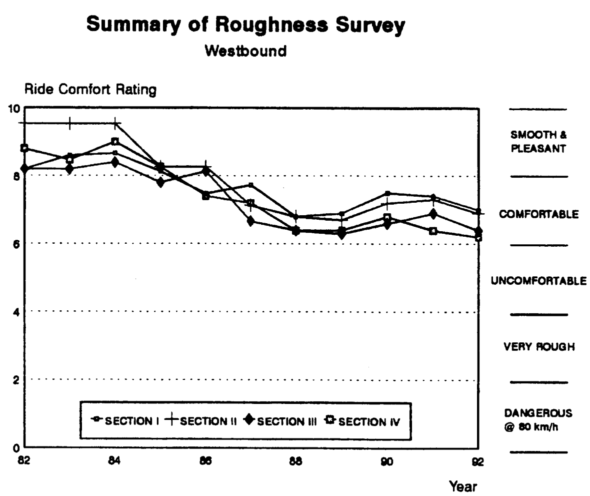

Chart 1

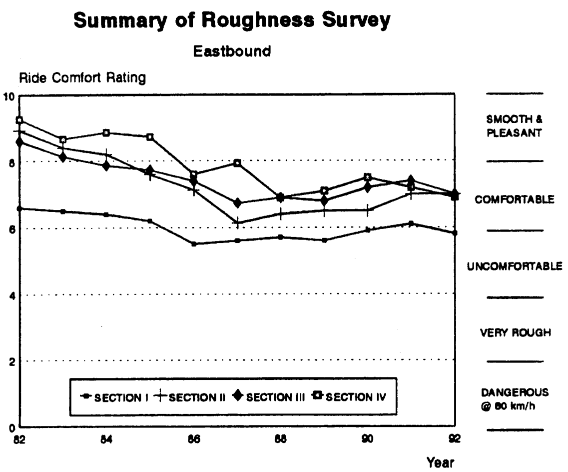

Chart 2

The RCR results over the past ten years for the westbound lanes have been consistent for all four sections. The RCR of the sections range from 8.2 to 9.5 in 1982 and from 6.2 to 7.0 in 1992. This represents an average drop in RCR of 20% to 25% in the westbound lane.

The results are similar for the eastbound lanes except for Section 1 where the roughness was only 6.6 in 1982 but remained quite constant over the last ten years with an RCR of 5.8 in 1992.

Overall the ride on Hwy. 3N after ten years of service is in the lower portion of the comfortable range. The decrease in the RCR over the last ten years is most likely due to the progressive faulting of joints and increased incidence of transverse cracking which is discussed later.

Skid Tests

The relative frictional resistance of a pavement is given in terms of a skid number. This number was obtained by field measurements using an ASTM brake force trailer (ASTM E274) and correlation with measurements from similar highway facilities.

The skid tests were performed each year on the burlap textured and astroturf textured sections in both the eastbound and westbound lanes. Results are shown in Charts 3 and 4.

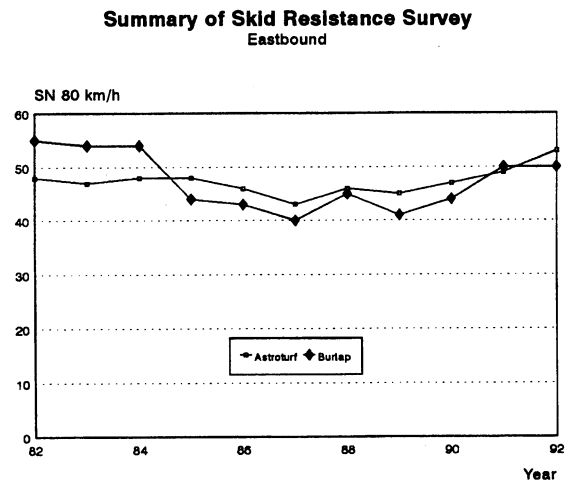

Chart 3

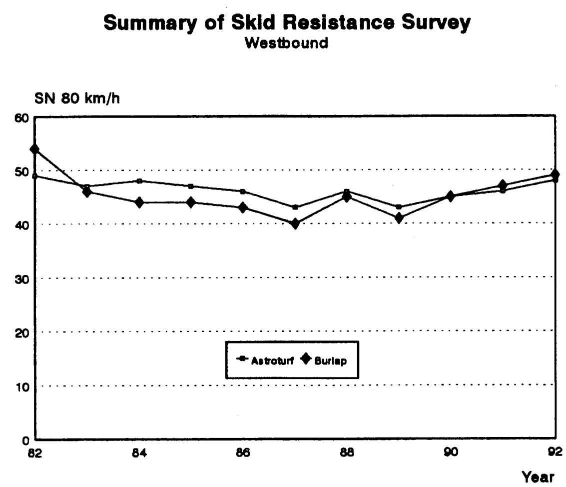

Chart 4

The magnitude of skid resistance with time for the two drag textures although initially higher for the burlap sections, are comparable with average Skid Numbers at 80 km/h (SN80) in the mid to low 40 range. As anticipated the sharp projections of the drag/tined surfaces, which resulted in high initial values, have been worn, polished or sheared off by traffic and snow removal operations.

Overall, these results indicate a high friction factor has been maintained, comparable to freeways with premium asphaltic concrete surface coarses utilizing quality aggregates.

Load Deflection Testing

In order to compare the load deflection characteristics and the joint load transfer efficiencies of the four pavement sections, three non-destructive load testing programs were conducted during the ten year monitoring period. For each pavement section, slabs were randomly selected for load testing using the Dynatest 8000 Falling Weight Deflectometer (FWD).

The tests were taken at the centre of the slab and at the leave and approach corner of the slab in order to determine the centre slab deflections and the load transfer efficiency at the joints.

Load Transfer Efficiency

The FWD testing was done in September 1985, November 1988 and July 1992. The statistical summation of the load transfer efficiency is shown in Chart 5.

Load Transfer Efficiency

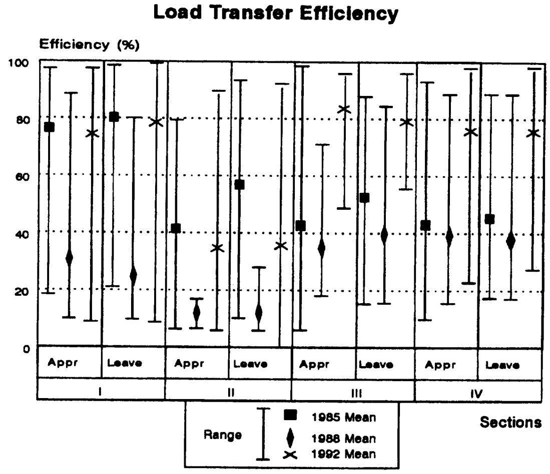

Chart 5

The results between the load transfer efficiency in the leave and approach slabs are very similar; the differences range from 1% to 12% for all slabs. For purposes of the discussion, the average between the approach and leave slabs will be used for the load transfer efficiency of each individual Section.

Section I shows very little change in the load transfer efficiency between the 1985 and 1992 results with an average of 77% in 1985 and 75% in 1992. The low reading of 27% in 1988 is most likely due to the cooler temperatures experienced during FWD testing in November.

Section II shows the least effective load transfer with low reading for all years of testing - 48% in 1985, 12% in 1988 and 30% in 1992. Even taking into account the warmer temperatures during testing in 1985 and 1992, the load transfer efficiency of this section is poor.

Section III and IV exhibit very similar load transfer characteristics. The percentage load transfer between 1985 and 1988 for both sections was within 10% but there was a significant increase between 1988 and 1992 - the efficiency approximately doubled for each section. This is likely due to the FWD testing in 1992 taking place in the early afternoon when pavement temperature was at its warmest. Section I and II were measured in the morning during cooler temperatures.

Overall, Section I is showing the highest level of load transfer efficiency, Section II the lowest, with Sections III and IV showing comparable moderate levels of efficiency.

The results to date reinforce the concept that under heavy axle loads in zones experiencing significant temperature fluctuations, relying on aggregate interlock at the joint provides only a marginal short term load transfer mechanism. As is presently occurring on Hwy. 3N, faulting of the joints is increasing, and the ride of the pavement is becoming progressively rougher.

Centre Slab Deflections

A statistical summation of the centre slab deflections is shown on Chart 6.

Centre Slab Deflections (40 kN Load)

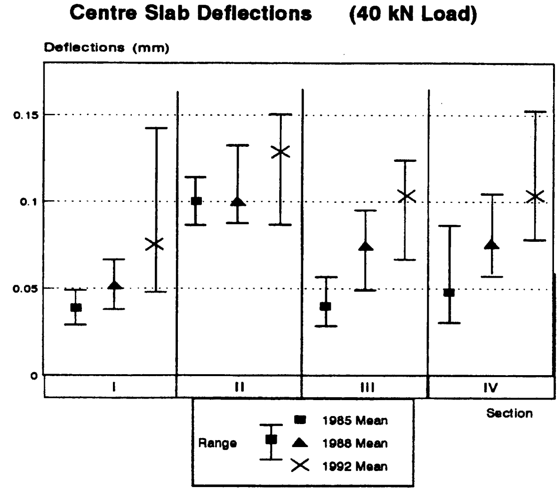

Chart 6

During the ten year monitoring period, mean centre slab deflections have increased by 0.04 mm for Section I, 0.03 mm for Section II, 0.07 mm for Section III, and 0.06 mm for Section IV.

Briefly the results indicate the full depth concrete is still exhibiting the greatest stiffness under load while the two sections on Lean Concrete Base (LCB) are experiencing a reduced level of rigidity when compared to initial readings. The section of OGDL continues to experience the highest centre slab deflections, indicative of the least rigid performance of the four pavement sections monitored.

Crack Survey

A detailed crack survey, using the methodology outlined in [8], was initiated one year after construction and updated in 1983, 1984, 1985, 1988, and 1992. The severity and density of these distresses were recorded and their annual growth noted.

A summary of the extent, number, type and density of cracking is shown in tabular form on Table 1 and the crack density (cracks per kilometre) against age for each section on Chart 7.

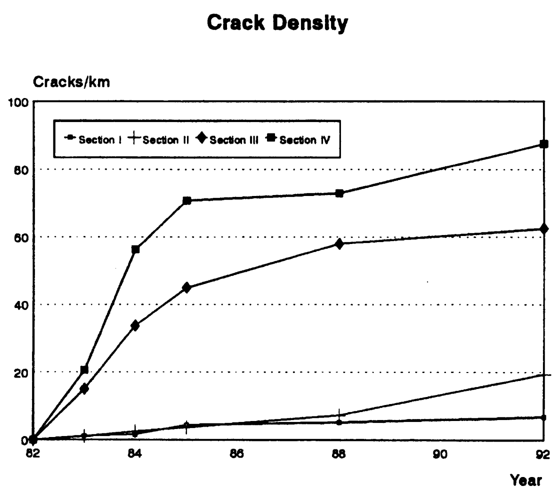

Chart 7

In Section I, the amount of cracking has remained at a fairly constant low level over the past ten years. The largest increase being in the amount of partial transverse cracks, slight to moderate in severity, increasing from 2 to 8 in the last four years, while the number of full width transverse and longitudinal cracks has remained constant since 1988.

Section II had a moderate increase in cracking over the period from 1988 to 1992. The incidence of full width transverse cracking has increased from 36 to 74 during this period. However, there is no incidence of longitudinal cracking. This increase in transverse cracking may be indicative of progressive localized contamination of the OGDL. This reduction in the ability of the OGDL to properly drain the pavement section has resulted in a loss of structural support. This phenomenon is discussed further under the OGDL investigation.

The crack performance of Section III has levelled off and remained relatively constant since 1985 with an increase from 29 to 39 for transverse cracks and an increase from 6 to 8 for longitudinal cracks in the same time period. However, the overall crack density of 62.5 cracks per kilometre is high when compared to Sections I and II.

Section IV remains the most severely distressed section with 88 cracks per kilometre, although the increase since 1984 has been fairly constant. The majority of the cracking took place in the first three years but has continued to increase over the past seven years. Possible causes of this cracking have been discussed in reference [1].

Timely and proper saw cutting of all joints and the use of a bond breaker between the slab and LCB would have significantly reduced the incidence of cracking in these two sections.

Some slight spalling is apparent at the more severe cracks for all sections. All major cracks noted in the 1983 survey were routed and sealed in early 1984, additional cracks were again sealed in 1986. The majority of new cracks since 1986 have not been sealed to date.

Faulting Measurements

Joint faulting (stepping) and isolated indications of mud pumping and staining was initially apparent and reported in 1987, some 5 years after construction. Results of 1988 and 1992 fault surveys are plotted on Chart 8.

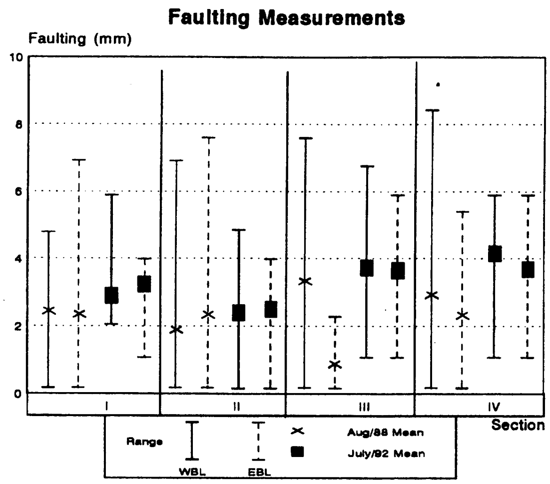

Chart 8

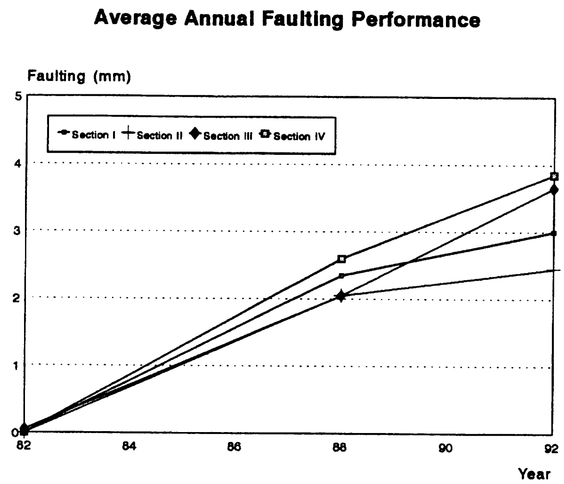

Chart 9

The average depth of the faulting in both the eastbound and westbound lanes for Sections I and II increased by 25% to 3.0 mm and 2.4 mm respectively from 1988 to 1992.

In Section III the average faulting in the westbound lanes increased by 12% (from 3.3 mm to 3.7 mm) and by 350% in the eastbound lanes (0.8 mm to 3.6 mm) during the same period.

Section IV had a 41% and 50% increase in the average faulting depth in the westbound and eastbound lanes respectively. Overall, Sections III and IV have experienced an average of 3.6 mm and 3.8 mm of faulting to date. The rate at which the faulting is occurring is very similar in Sections III and IV at 0.36 mm/yr and 0.39 mm/yr respectively. Sections I and II have annual rates of faulting of 0.31 mm/yr and 0.25 mm/yr respectively. The average annual fault performance is plotted on Chart 9.

OGDL Investigation

In order to evaluate the recent condition of the OGDL, cores were taken in June of 1989. For a detailed review of the coring investigation, see [9]. The core investigation revealed that:

1) Cores taken at mid-slab, away from any cracks, revealed that the bottom 20 mm of the OGDL was contaminated with the silty clay subgrade material and was showing slight signs of stripping. The AC content at the bottom of the core was 2.0% compared with 2.7% at the top. The OGDL itself was moist and highly permeable.

2) The cores taken at faulted transverse joints near the centre of the lane and the outside wheel path revealed loose, partly stripped OGDL aggregate heavily contaminated with subgrade material. The AC content ranged from 1.1% to 1.9%. The OGDL material in all core holes was saturated, with drill water not dissipating away from the core holes.

3) The outlets appeared to be clean and in good condition. The rate of water discharge appeared to be low and about the same for all sections.

This investigation confirmed that the OGDL was poorly drained, resulting in ponding of water within the OGDL (i.e. a "bathtub effect"), saturating and softening the silty clay subgrade, and accelerating the movement of subgrade fines into the OGDL.

Performance of Section II reaffirms the design requirements of having the collector system in direct contact with the OGDL to maximize drainage away from the pavement structure and to ensure a properly designed filter separates the OGDL from the pumpable subgrade [10].

Neither of these requirements were achieved on Hwy. 3N. The plowed subdrain collector system was backfilled with a less permeable, subgrade contaminated, granular base aggregate and the OGDL was placed directly on a pumpable silty clay subgrade.

Sound Level Measurements

In order to assess relative sound level characteristics of the burlap versus astroturf drag surface textures, sound level measurements were taken in the early life of the project. Three testing procedures were utilized for comparison purposes. The first procedure is a near tire method comparing noise from pavements. The second method measures the maximum sound levels of a vehicle passing by 15 m from the centre of the travelled lane. The third method measures noise due to the total traffic flow at two different locations adjacent to different pavement types. The comparative results of the three methods are detailed in [1].

In the near tire tests (Method 1) there was little difference between the noise emitted by the two different concrete pavement surfaces. The concrete was only marginally louder (1 dBA) than a nearby hot mix pavement. During roadside testing using Methods 2 and 3 the difference between the pavements was not statistically significant.

Overall, the burlap and astroturf drags resulted in a pavement texture with little or no appreciable difference in sound levels under normal vehicular traffic conditions. However, slight tonal differences are apparent between the two surface types.

CONCLUSIONS

This report summarizes the results of ten years of performance observation on four experimental concrete pavement sections. Data collection is continuing and performance results are continually reviewed.

The following general conclusion based on these results appear warranted:

1) The Ride Comfort Rating for all four sections has experienced an average drop of 20-25% over the ten years monitored. This decrease in ride, to the lower portion of the comfortable range, is a result of the progressive slight to moderate faulting of joints and increasing incidence of cracking.

2) Skid values have stabilized for all sections at a high level of serviceability. There appears to be no significant difference in the skid resistance, ride value, or noise levels between the burlap and astroturf drag texturization treatments.

3) Poor load transfer efficiencies coupled with the progressive faulting experienced at all joints indicate the undowelled joint design was inadequate for the much higher than estimated traffic volumes experienced. The full depth section is experiencing the highest level of load transfer, the OGDL section the lowest, and the LCB sections have comparable levels of load transfer efficiency.

4) Based on centre slab deflection measurements all sections have experienced increases in deflection readings (0.03 to 0.07 mm); Section I still exhibits the highest stiffness and Section II the least of the four sections.

5) The incidence of cracking has stabilized in most sections, with the highest increase in cracking occurring in the first three years. Both LCB designs continue to exhibit the poorest distress performance, with a high incidence of transverse and longitudinal cracking. Of interest is the recent moderate increase in occurrence of transverse cracking in the OGDL design section.

6) Faulting is occurring at an average annual rate of 0.25 mm to 0.39 mm per

year depending on the section monitored. Section IV is experiencing the highest average level of faulting (3.8 mm) and Section II the lowest (2.4 mm), even though Section IV has the highest sectional modulus and Section II the lowest sectional modulus.

7) The field investigation indicates the OGDL in Section II is poorly drained and contaminated with silty clay subgrade material. These results reinforce the need for the OGDL to be designed and constructed as part of a total pavement drainage system utilizing a free flowing collector system in direct contact with the OGDL and a protection (filter) system between the pumpable subgrade and the OGDL.

In summary, much information and experience has been gained and continues to be gathered based on the performance of the Hwy. 3N experimental concrete pavement sections over the past ten years. This knowledge has been applied to develop improved design guidelines and revised specifications to ensure the long term performance and serviceability of our recently constructed concrete pavements in Ontario's wet-freeze environment.

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Summary of Annual Distress Survey

Pavement Section	1983 Survey No. of Cracks				1984 Survey No. of Cracks				1985 Survey No. of Cracks				1988 Survey No. of Cracks				1992 Survey No. of Cracks			
Distress Type	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
Transverse Cracking																				
-Partial Lane Width	--	--	--	--	--	--	3	14	2	--	3	22	2	3	3	25	8	2	3	39
-Full Lane Width	5	--	11	74	6	--	18	202	6	3	26	240	6	15	36	244	6	74	36	271
Longitudinal Cracking	--	--	1	2	--	--	5	25	4	--	6	41	8	--	6	45	8	--	8	62
Accummulated Length, metres	--	--	15	107	--	--	81	322	26	--	86	505	33	--	89	600	35	--	112	702
Corner Cracking	1	4	--	16	2	9	--	24	6	10	--	30	6	14	--	30	11	14	2	41
Joint Spalling	--	1	--	5	--	4	1	5	3	6	1	7	3	6	1	7	3	7	1	7
Total No. of Cracks per Section	6	5	12	99	8	13	27	270	21	19	36	340	25	38	46	351	34	97	50	420
Crack Density																				
Cracks per kilometre	1.2	1.0	15.0	20.6	1.6	2.5	33.8	56.3	4.4	3.7	45	70.8	5.2	7.3	58	73	6.8	19.4	62.5	87.5

Table 1

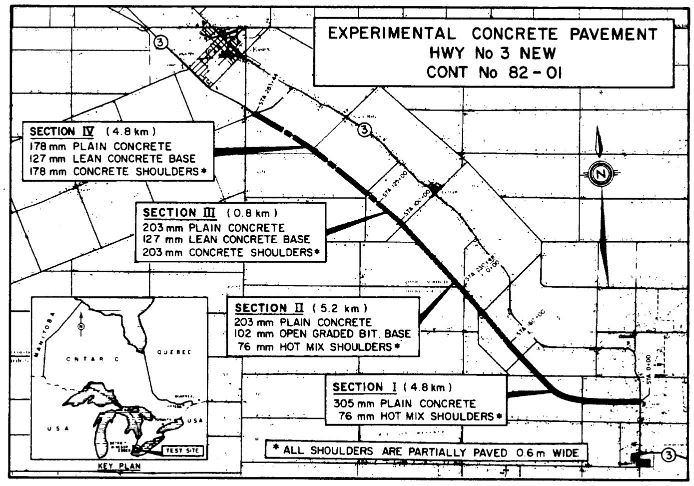

Figure 1

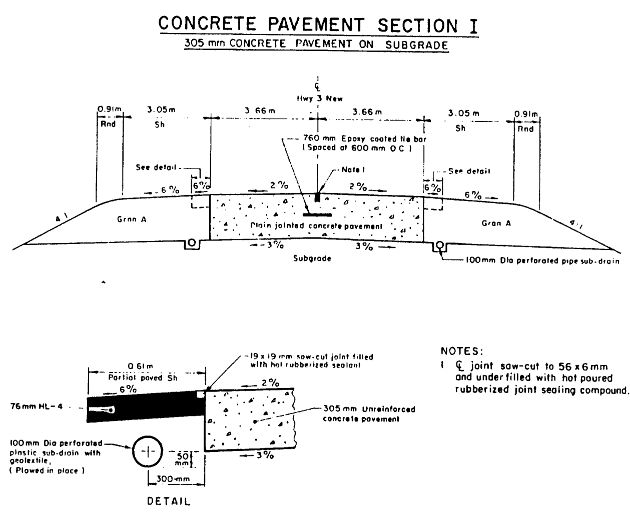

Figure 2

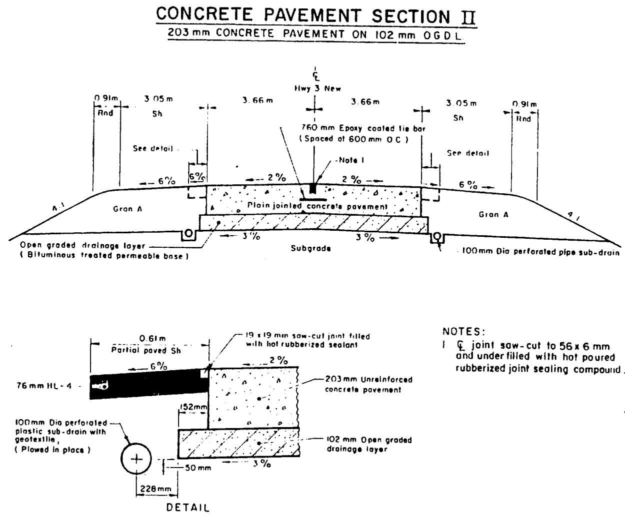

Figure 3

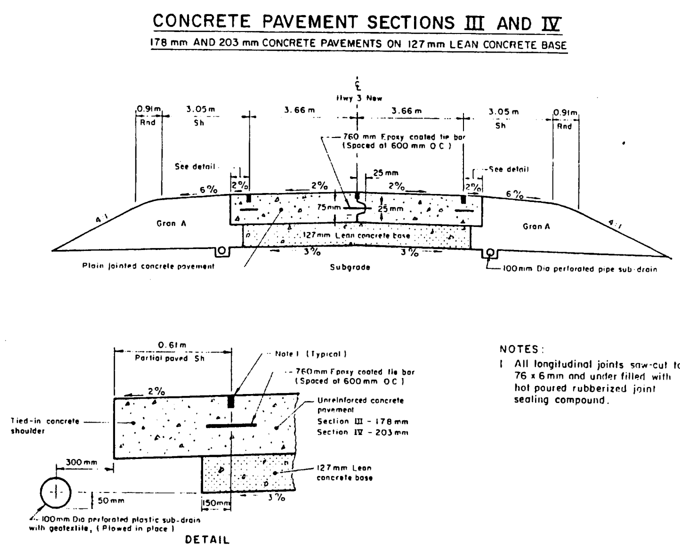

Figure 4