"SIX YEARS EXPERIENCE WITH EXPERIMENTAL CONCRETE PAVEMENT SECTIONS IN ONTARIO"

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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 pavement, drainage, and shoulder designs plus 2 types of surface textures.

Recent innovative developments in concrete pavement design methodology, material specification, construction techniques, and pavement drainage systems prompted Ontario's Ministry of Transportation to construct these alternative test pavements. A summary of the design and construction details, plus the performance results of an ongoing monitoring program are documented in this paper.

The performance of the pavement sections is described in terms of load transfer and pavement edge deflections based on FWD testing, Pavement Condition Ratings, roughness, skid resistance, joint movement and a crack survey. Observations of noise levels, traffic volumes, and surface texture are discussed.

Conclusions based on six years of performance results indicate the superior performance of the free draining base materials. In addition, some anomalous behaviour based on pavement cracking and roughness suggest additional areas of process control are warranted.

INTRODUCTION

In 1982, Ministry of Transportation of Ontario constructed a 15.6 km (9.7 mile) length of experimental concrete pavement which incorporated varying thicknesses of plain jointed undowelled concrete pavement 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:

(I) 4.8 kilometres (3 miles) of 305 mm (12 inch) thick plain jointed undowelled concrete with 76 mm (3 inch) thick hot mix shoulders

(II) 5.2 kilometres (3.2 miles) of 203 mm (8 inch) thick plain jointed undowelled concrete with 76 mm (3 inch) thick hot mix partially paved shoulders over 102 mm (4 inch) open graded drainage layer (bituminous treated permeable base)

(III) 0.8 kilometres (0.5 miles) of 203 mm (8 inch) thick plain jointed undowelled concrete with 203 mm (8 inch) tied-in plain concrete shoulders over 127 mm (5 inch) lean concrete base

(IV) 4.8 kilometres (3 miles) of 178 mm (7 inch) thick plain jointed undowelled concrete with 178 mm (7 inch) tied-in plain concrete shoulders over 127 mm (5 inch) 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.

Upon completion, the following performance features of the experimental pavement were measured:

Pavement Thickness
*Roughness (*denotes being continually monitored)
*Skid Resistance
*Load/Deflection Characteristics
*Pavement Condition Surveys
*Crack Survey
*Joint Movement
Subgrade Moisture Content
Noise Levels
*Traffic Volumes

This paper summarizes these measurements at the completion of construction and over the next 6 years, discusses observations about the performance and serviceability of the various designs, and presents recommendations based on these short term results.

ONTARIO PERSPECTIVE

Concrete pavements constructed in Ontario prior to 1968, were designed as 230 mm (9 inch) thick reinforced and nonreinforced concrete slabs on 230 mm to 300 mm (9 inch to 12 inch) granular subbases, with dowelled joints at spacings which were successively reduced from 30 m (99 ft.) in 1958, 21.3 m (70 ft.) in 1960, 17 m (56 ft.) in 1963, to 8.8 m (29 ft.) in 1967. [1,2] The major problem encountered with these designs was the increasing incidence of transverse cracks and subsequent development of spalling and faulting at joints and cracks which caused deterioration in ride quality. No cost-effective maintenance treatment for these problems was then available, so thick asphalt overlays were used to rehabilitate these pavements at ages which varied from 12 to 16 years, with a few sections reaching 20 years. Between 1968 and 1971, concrete pavements were designed as plain 9 inch slabs with dowelled skewed joints at random spacings of 4, 5.8, 5.5 and 3.7 m (13, 19, 18 and 12 ft.) placed on a 127 mm (5 inch) cement stabilized or weak concrete base. These latter designs are still performing remarkably well under extremely heavy truck traffic. However their high initial cost makes this design difficult to justify, and escalating rehabilitation costs during the ensuing years did not make justification easier. No new concrete pavements were built during the years 1971 and 1982.

As encouraging reports of improved performance of other rigid pavement designs with tied concrete shoulders and non- erodible bases were received from the USA and Europe, it was decided that Ontario needed to explore these new ideas, in combination with their own experiences, to find promising new lower-cost designs. Subsequently, the decision was made to construct and monitor four experimental concrete pavement sections on Hwy. 3N.

DESIGN PARAMETERS

The experimental pavement was designed as a 7.3 m (24 ft.) wide, two lane new facility project with 0.6 m (2 ft.) partially paved shoulders, located some 24 kilometres (15 miles) south west of Windsor in the Township of Gosfield North and South, County of Essex. Refer to site plan, Figure 1. All pavement tapers, speed change lanes, connected 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. Physiographically, the site is located on a glacio-lacustrine clay plain, with the southern limit located on a sand plain. The topography is flat over most of the alignment with some relief at the south end. The subgrade soils consist predominately of silty clays (CBR of 5) of low to intermediate plasticity (CL-CI) overlain by sands, silty sands and gravels in the southern 1.6 kilometres of the alignment.

The average annual daily traffic (AADT) anticipated in 1978 was 4,400, with 20 year projected AADT of 8600. Thirteen percent of this traffic is classified as commercial traffic. This represents approximately 3.5 million (80 kN) equivalent single axle loads (ESAL) applications over the 20 year design period.

Typically, frost penetration in this area ranges from 760 to 910 mm (30 to 36 inches), with a freezing index of 400-500 degree days.

In consideration of these parameters a benchmark design of 203 mm (8 inch) thick plain jointed undowelled concrete pavement over 127 mm (5 inch) lean concrete base was selected as an alternative to the initially approved 305 mm (12 inch) full depth bituminous design. The alternatives selected represent variations in design strategy utilizing different erodible bases and subbases [3] while maintaining similar strength equivalences. Detailed sections of the four pavement designs are shown on Figure 2, 3 and 4.

In consideration of the anticipated light truck traffic condition, it was felt that sufficient load transfer between slabs would be achieved through aggregate interlock by closely spaced undowelled joints. To insure a smoother ride over the joints and improved joint performance a randomized pattern of 4, 5.8, 5.5, 3.7 m (13, 19, 18, 12 ft.) for transverse joints skewed at 1:6 was recommended.

Joints were sawn to a width of 13 mm (0.5 inch) and a depth of 25 mm (1 inch), blown clean and filled with a hot poured rubberized sealant.

MATERIALS AND CONSTRUCTION

Details of the construction and records of material testing have been published in an earlier report [9]. In general, current M.T.O. specifications for grading, granular base, hot mix, lean concrete base and concrete pavement were applied, without change. The following special provisions were included in the contract to deal with specialized construction materials, methodology and test procedures as required for the experimental pavements:

To insure the subgrade would perform uniformly and allow rapid drainage flow into the subdrain systems, the use of an impermeable selected clay borrow was specified between Sta. 34+00 and Sta. 160+00.

Upon final shaping and compaction of the subgrade, construction haul was restricted to shoulder use.

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

Drainage Layer

The Open Graded Drainage Layer (O.G.D.L.) is basically an open textured, uniformly graded, A.C. stabilized coarse aggregate which was placed in a 100 mm (4 inch) 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 coarse aggregate for the bituminous treated O.G.D.L. was specified as 100% crushed material. The O.G.D.L. was designed as a modified HL 8 asphaltic concrete with a 90/10 coarse aggregate/fine aggregate split and 2% by weight of an 85-100 penetration asphalt cement binder. This ratio was slightly modified during construction to improve stability. Typical gradation curves for the O.G.D.L. mix are shown on Figure 5, adapted after Cedergren [4].

Field permeability testing indicated a lateral flow velocity of 2.8 x 10^{-1}cm/sec (800 ft/day). Based on laboratory permeability testing of cored field samples the average vertical permeability = 1.2 x 10^{-1}cm/sec (350 ft/day). 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.

Provision was made for two days of evaluation and testing upon completion of the initial kilometre of concrete pavement, to insure material quality, surface tolerance, and texture had been achieved.

A 100 mm subdrain wrapped with a knitted filter sock was installed to the required grade and location by ploughing [5,6] and reinstating the displaced granular material in one continuous operation.

In order to insure 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 350 freeze/thaw cycles) as per ASTM C666, Procedure A were specified. The coarse aggregate used on site was a Manitoulin Island Dolomite.

The specification requirements and results of quality assurance acceptance testing for lean concrete base and concrete pavements are summarized in chart form on Figure 6.

The lean concrete base was constructed by conventional methods using slipform paving equipment to a total width of 7.6 m (25 ft.), ie. 150 mm (6 inches) beyond the edge of concrete pavement. The lean concrete was centrally mixed on site, transported by end-dump and belly-dump trucks, and placed on the prepared base by a mechanical spreader.

The concrete wearing course was constructed using conventional slipform paving techniques. Additional offsite ready mix concrete was used for the full depth slab, Section I, to supplement the production of the site plant. Concrete placement commenced on 82 09 18 and continued to completion of 82 10 21 with an average production rate of 1530 cubic metres (2000 cubic yards) per day. The sequence of pour consisted of beginning at the east end (Sta. 0+11), slipforming both 305 mm full depth section and the 200 mm of O.G.D.L. (both with 0.6 mm asphalt shoulders) for the full 7.3 m (24 ft.) width. The final two sections with tied-in concrete shoulders were slipformed in one lane plus shoulder (3.7 m + 0.6 m) width, starting with W.B.L. travelling westerly and then shifting to the E.B.L. and travelling easterly.

COST COMPARISON

Section	Length km (miles)	Pavement Structure	Cost (1981) $/km ($/mile)
I	4.8 (3.0)	- 305 mm P.C. Concrete	$ 220,300 (354,600)
II	5.2 (3.2)	- 203 mm P.C. Concrete 100 mm O.G.D.L. 76 mm Asphalt Shoulder	$ 218,700 (351,900)
III	0.8 (0.5)	- 203 mm P.C. Concrete 127 mm Lean Concrete Base 203 mm Concrete Shoulder	$ 272,600 (438,700)
IV	4.8 (3.0)	- 178 mm P.C. Concrete 127 mm Lean Concrete Base 178 mm Concrete Shoulder	$ 260,600 (419,400)

INSTRUMENTATION AND MONITORING

Profilograph Measurements

Immediately upon setting of the concrete, a California Type Profilograph was walked through the pavement wheelpaths in the direction of paving. For a description and operation details of a profilograph, refer to [7]. A summary of the profilograph test results for 1,000 and 4,000 feet sections plus each day's paving is presented in Figure 7. A complete breakdown of the profile index data measured is given in the appendix of [7].

Roughness Measurements

A commonly used technique for measuring pavement roughness is the Mays Ride Meter (MRM).

This unit consists of a towed trailer which has been instrumented to measure the vertical movement of the axle in relation to the vehicle or trailer's body. The chart recorder provides a continuous recording of the vertical movement of the unit's body and the distance travelled by the unit. The chart recorder paper feed is dependent upon the vehicle axle/body movement and advances 1/64 inch for every 0.1 inch of axle/body movement.

The measured movement of the vehicle is expressed as inch of axle/body movement per distance or inch per mile (I.P.M.). The Mays Ride Meter accumulates the axle/body movement of the unit through a photocell and celluloid vernier measure.

Each pavement section was tested three times during the initial 2 years of service and then on an annual basis. Tests were carried out in each lane at 80 km/h and a plot of average annual roughness plus summary is given on Figures 8 and 9.

Skid Testing

In North America the most commonly accepted technique for measuring pavement skid resistance is based on the use of a skid trailer conforming to ASTM Standard E274. The test unit consists of a towing vehicle and a Brake-Force Trailer. The test measures the steady friction force generated when the locked left test wheel standard tire slides over the wetted pavement at a constant travelling speed. During the test a specified quantity of water from the on-board supply system is discharged through a nozzle in front of the test tire to produce a water film thickness of 0.5 mm. The standard (ASTM 501) tire on the brake-force trailer has seven plain ribs on the tread. The measured friction force is described as the skid number (SN) at a specific speed, SN80 at 80 km/h for example.

Each pavement surface type was tested three times during the first 2 years of service, twice in 1985, and subsequently on an annual basis. Tests were carried out in the left wheelpath of each lane at 80 km/h and a detailed plot of the skid results is shown on Figure 10 for the westbound lanes and Figure 11 for the eastbound lanes.

Load Deflection Testing

In order to compare the load/deflection characteristics and the joint load transfer efficiencies of the four pavement sections, two non-destructive load testing programs were conducted. For each pavement section, slabs were randomly selected for loading testing using the Dynatest 8000 falling weight deflectometer (FWD). This FWD consists of a mass that can be dropped onto the pavement from several heights to simulate varying axial loadings, an electrical load cell to measure the transmitted load to the pavement, and seismic sensors to measure the displacement of the pavement at seven locations (deflection basin). This instrumentation is mounted on a trailer and the data signals are fed to a HP 85 computer mounted in the tow vehicle. The deflection measurements required to calculate load transfer for the approach and leave sides of the joint were obtained simultaneously with the corner slab measurements by "straddling" the joint using a trailing sensor. The majority of the testing was performed early in the day to minimize the influence of slab curl resulting from differential slab temperatures. No change in the transverse joint opening (13 mm) was apparent at the time of testing.

Typically, the test sequence consisted of the mass being dropped four times; initially to seat the load plate, and then from three different heights corresponding to three load levels, resulting in three sets of deflection readings for each sensor. These were then used to plot the load/deflection characteristics of each pavement type.

Sound Level Measurements

In order to assess relative sound level characteristics of the burlap verses astroturf drag surface texture, sound level measurements were taken by our Research and Development Branch. Three testing procedures were utilized for comparison purposes:

The first procedure is a near tire method comparing noise from pavements. A microphone is placed 10 cm behind the tire of a testing trailer and the sound pressure level, in dBA, is measured while the vehicle is travelling 80 km/h. The advantage with this method is, that the sound levels are repeatable with different vehicle runs on different days. The ambient temperature will affect the absolute sound levels but not the relative levels measured between different pavements. The disadvantage of this method is that the sound levels are only measured behind the tire and underesti-

mate the sound being generated by the tire side walls.

The second method measures the maximum sound level of a vehicle passing by 15 m from the centre of the travelled lane. A single vehicle is tested on several different pavements at the same speed (80 km/h) and any differences should be simply due to different pavement/tire noise. The disadvantage is, that only one particular vehicle is considered and there may be differences between different types of vehicles.

The third method measures noise due to the total traffic flow at two different locations adjacent to different pavement types. If there are a large number of heavy vehicles the differences in pavement type can be masked by noise from the truck engines and the vehicles may not operate the same as when they pass the different microphone positions.

The results are expressed in the following table:

Summary of Results

Testing Method	Pavement Type	Sound Level (dBA)
1	Astroturf Concrete	102.3
1	Burlap Concrete	102.5
1	Old Hot Mix	101.8
1	New Hot Mix	102.1
1	Astroturf Concrete	103.3
1	Burlap Concrete	103.2
1	Old Hot Mix	102.5
1	New Hot Mix	102.7
2	Burlap Concrete	74.8
2	Old Hot Mix	75.3
3	Burlap Concrete	66.3
3	Old Hot Mix	67.3

Horizontal Reference Pins

In order to assess horizontal joint movement for the various pavement sections, four sets of pins were embedded in each section across two joints. These pins were placed across the joints between the 5.8 m and 3.7 m panels. Four sets of readings were taken in the first year following construction and two sets in every subsequent year.

To date, only seasonal variations are apparent in the data, with no appreciable change occurring in joint width spacing.

Moisture Gauges

To assess the effectiveness of the O.G.D.L. in improving drainage and reducing the moisture content at the pavement subgrade interface, two TDR type moisture gauges were installed at the edge of pavement to a depth of 390 mm, (ie. within the subgrade) in Section I and II. Readings were being taken several times per year, with data reflecting seasonal fluctuations but little annual gross change in moisture contents.

Crack Survey

A detailed crack survey, using the methodology outlined in [8], was initiated one year after construction and updated at regular intervals. The severity and density of these distresses were recorded, and their annual growth noted. In addition, to assess the extent of the distress and determine their causes, both the transverse and longitudinal cracks were selectively cored and analysed.

PAVEMENT PERFORMANCE

Ride Quality

The Hwy. 3N test pavement sections were profilographed along the four wheel paths in the direction of paving, with test results being summarized in Figure 7. Typically, the ratio of the mean Profile Index for the two lane versus one lane paving was 2:1 (for 4,000 ft. section) ranging from a mean of 9.75 to 4.93 inches/mile respectively.

In the two lane construction (Section I & II) 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 single lane construction (Section III & IV) where only 2 wheelpaths are paved, there is no clear distinction between the first set of wheelpaths (No. 1 & 2). However, in the second set of wheelpaths (No. 3 & 4), the inner wheelpath is consistently smoother (with some exceptions) than the outer wheelpath. This would be indicative of the problems; particularly on two lane width paving, associated with a paver controlled by a single grade control, except on the second single lane construction (Section III & IV, E.B.L.) where a travelling ski plus stringline were utilized.

In addition, 13 bump locations, low or high spots 6 mm (.25 inch) or greater in 3 metres (10 feet), were identified and verified using straight edge test results. These bumps were then reduced by grinding.

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

Roughness

A summary of the result of May's Ride Meter testing is shown on Figure 8 for the westbound lanes and Figure 9 for the eastbound lanes. The following 6 year trends are apparent:

1) Overall the eastbound lanes have a higher average roughness value (97 IPM) as compared to the westbound lanes (85 IPM), a difference of some 12%.

2) The E.B.L. of Section I has the highest average roughness (118 IPM) while the W.B.L. of Section II has the lowest (79 IPM), these sections had both lanes paved simultaneously. One explanation for this is the greater variability of grade control as you get away from the single paving guide (outside W.B.L.) compounded by paving on subgrade constructed with less tolerance than a machine laid O.G.D.L. surface.
3) Only in Section IV are the E.B.L. roughness values less than the W.B.L. values (85 vs 87 respectively). This is indicative of the improved surface tolerance achieved by twin grade controls (ie. travelling ski and stringline) used when slipforming the E.B.L.
4) Due to the short length of Section III, no interpretation or comparison of the data is presented.

The May's Meter Instrumentation has been shown by several authorities to be more susceptible to transversely grooved concrete pavements than hot mix bituminous pavements. From experience, newly constructed portland cement concrete pavements range from 70 to 100 IPM while newly constructed bituminous pavements ranged from 30 to 65 IPM. It appears that Hwy. 3N is comparable to previous experience with the exception of the eastbound 305 mm plain concrete section whose value is slightly above the norm.

Overall the present roughness of the concrete pavement sections can be described as ranging from good to slightly rough but predominately slightly rough throughout. This increased roughness is attributed to progressive slight faulting of the pavement joints.

Skid Resistance

The results of skid trailer testing on the 2 concrete surfaces are detailed in Figures 10 and 11 and summarized on Figure 12.

The magnitude of skid resistance with time for the astroturf and burlap drag textures, although initially higher for the burlap sections, are compariable (average SN80 equals 46 and 47 respectively).

The initial surface texture produced by the burlap drag and astroturf technique developed very sharp 0.5 mm projections at the lip of the transverse tines. This also occurred at the Hwy. 401 Morningside Project were the burlap drag technique was used in conjunction with a vibrating head plastic concrete grooving machine [10].

In the first few years of service at the Morningside Project these sharp 0.5 mm projections were either worn or sheared by traffic or snow removal procedures. This elimination of the projections drastically reduced the skid resistance. The rate of reduced skid resistance is heavily dependent upon traffic volume, the number of snow plough passes, and the aggregate mineralogy.

At this Hwy. 3N location, it appears that the rate of reduction of skid resistance is much slower in comparison to the 401 projects due to lower traffic volumes and lighter commercial vehicle loadings.

Load Response Characteristics

A statistical summation of the centre slab deflections and load transfer efficiencies under a simulated 40 kN load for both the Sept./85 and Nov./88 FWD results are shown in tabular and graphical form on Figures 13A and 13B.

During the 3 year period between the two sets of readings, mean centre slabs deflections have increased by 35% for Section I, 83% for Section III and 65% for Section IV, but have remained the same for Section II.

Briefly, the results indicate the full depth concrete design (Section I) and the 203 mm concrete on lean concrete base (Section III) continue to exhibit similar performance under load, with Section I demonstrating the greatest stiffness under the maximum allowable single axle loadings.

Of notable interest, is the apparent higher deflections, and consistency of deflection results, experienced by the concrete pavement on O.G.D.L. (Section II) as compared to Section I. From a design point of view, Sections I, III & IV are exhibiting similar structural stiffness characteristics with time, however, Section II is exhibiting significantly less stiffness and change in stiffness with time.

The results of load transfer testing are interesting, although the range of readings are large. Section I initially had the highest mean load transfer efficiencies (74 to 81%), however current readings indicate a drastic reduction (29 to 25%) over the three year period. Similarly, Section II with the lowest mean readings (42 to 54%) dropped significantly (12 to 13%). Sections III and IV also experienced reductions in load transfer efficiencies, although not of the same magnitude.

The initial improved load transfer characteristics of the full depth section which included increased aggregate interlock due to larger contact area and the inertia of the thicker slab have been rendered less effective with time.

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, large deflections at the joint are wearing out this aggregate interlock, faulting of the slabs is developing and the ride of the pavement is becoming progressively rougher.

Sound Levels

As previously detailed, three methods of testing for sound levels were utilized and the results presented.

In the near tire tests (Method 1) there was almost no 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 (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.

DISTRESS MANIFESTATION

Overall, two significant distress types generally associated with structural deficiencies have occurred within the 6 year monitoring period:

1) the widespread incidence of cracking, both transverse and longitudinal, in Sections III & IV founded on lean concrete base.
2) the premature development of slight joint faulting in all the sections.

Cracking

A summary of the extent, number, type and density of cracking is shown in tabular form on Figure 14.

Briefly, the full depth concrete section and the O.G.D.L. section has undergone negligible amounts of cracking. The former consisting of slight corner cracking and moderate full lane transverse cracking only on the high side of the superelevated curve (indication of shrinkage performance). The latter, Section II, has no longitudinal cracking whatsoever and only a minimal occurrence of transverse cracking. Distresses noted consist of slight to moderate corner cracking and joint spalling, associated with joint performance. In addition, both sections show little increase in crack density over the 6 year survey period, as shown graphically on Figure 15.

In the Sections with lean concrete base, significant amounts of cracking have occurred. Shrinkage cracks, both full or partial width transverse cracks, appeared at the surface of the base before surfacing was applied. After surfacing, partial and full width transverse cracks and longitudinal cracks appeared in these sections and dramatically increased in frequency and length during the first 3 years of survey. The longitudinal cracking, slight to moderate, generally runs parallel and some 0.6 to 1.0 metres either side of centreline. In places severe stepping of the longitudinal cracks is already apparent. Most of the cracking in these two sections was initially of the hair-line variety, but rapidly widened. Some slight spalling is apparent at the more severe cracks. The crack density (cracks per kilometre) for each section is shown on Figure 15. All major cracks noted in the 1983 survey were routed and sealed in early 1984, and additional cracks again sealed in 1986.

To better assess the possible causes of cracking in Sections III and IV, a total of twenty 150 mm diameter cores were taken including transverse and longitudinal cracks, and longitudinal centreline and shoulder joint. Visual examination and analysis of the cores indicate the following:

1) At the transverse locations; the crack is full depth, passing both through the paste and aggregate of the concrete pavement but only through the paste in the lean concrete base. This is indicative of reflection cracking up through the base since the crack had occurred after the concrete pavement had hardened.
2) The longitudinal cracks appear to be the result of shallow and delayed cutting of the longitudinal centreline joint in combination with some reflection cracking, since some cores show full depth cracking while others show no cracking in the lean concrete base.

Faulting

Joint faulting (stepping) and isolated indications of mud pumping and staining was initially apparent in 1987, some 5 years after construction. Results of the 1988 survey, as plotted on Figure 16, indicates mean faulting of the joints in all sections to range from 2.0 mm (0.08 inch) to 2.7 mm (0.11 inch) with individual joint values ranging from nil to 8 mm (0.3 inches). These measurements were taken at the outside slab corner, where typically the greatest vertical displacements have occurred.

No differences in the magnitude of faulting were measured in either traffic direction.

Although not significantly affecting the ride quality of the pavement at the present time, this premature occurrence of faulting is indicative of load transfer deficiency across the joints and possible future loss of slab support.

Other Distresses

The partially paved asphalt shoulders on Section I & II were depressed and exhibited severe pavement edge drop-off in the order of 10 to 35 mm (0.5 to 1.5 inches). In addition, the asphalt shoulders exhibit both transverse and alligator cracking. At these locations where the transverse joints intersect the pavement edge, a high inci-

dence of slight spalling of the joint (up to 75 mm in length) is observed. Pumping has resulted in isolated blowhole formation in the asphalt shoulder.

Although the hot paved rubberized sealant is 99% intact, it is depressed within the joint reservoir and exhibits little or no bond with the joint face.

The pavement shows no signs of surface or concrete durability deficiencies.

Performance after Six Years of Traffic

Traffic readings taken in the spring of 1988 indicates 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 3 years and well beyond the 1998 (20 year) projected AADT of 8600 used in design.

The actual accumulated number of ESAL in 1988 is 1.0 million as compared to the predicted 6 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.

At present the pavement is assigned a condition rating in the mid 80's and an acceptable ride comfort rating representative of generally good performance. However, the magnitude of cracking in Sections III and IV plus the appearance of measureable faulting is becoming a concern.

CONCLUSIONS

This report summarizes the results of 6 years performance observation on four experimental concrete pavement sections. Data collection is continuing and performance results are continually reviewed.

The following general conclusions based on these results appear warranted:

(a) Both the full depth and O.G.D.L. designs are showing a comparable high level of performance and serviceability to date. This is of significant note since, based on their immediate structure response to loadings and the resultant measured deflection, the O.G.D.L. design has the lowest sectional stiffness and the full depth design the highest.

(b) The two sections (III and IV) with lean concrete base, although more costly to construct, have the poorest distress performance of the four sections. Both exhibit a high incidence of full width transverse and longitudinal cracking which has increased considerably over the observation period.

(c) Based on the poor load transfer efficiencies measured, the recent occurrence of slight joint faulting, and the unexpectant increase in traffic volumes; the undowelled joint design appears inadequate for the conditions experienced.

(d) The highest mean load transfer efficiencies were achieved in the full depth concrete section, while the other 3 sections obtained similar fair results. Future plain jointed concrete pavement designs should maximize the diameter of coarse aggregate, minimum 40 mm diameter but never greater than 1/3 the depth of the slab. In addition, consideration should be given to specifying aggregates with an angular shape and rough texture to maximize the bond between cement paste and aggregate. Both these measures will improve aggregate interlock and durability across joints and insure a higher degree of load transfer efficiency across non-dowelled joints.

(e) There appears to be no appreciable difference in ride quality, skid resistance, and noise levels between the burlap or astroturf drag texturization treatments as noted in the short term.

(f) Future concrete pavements should be constructed with concrete shoulders to minimize shoulder distress, differential performance, and pavement drop off. In addition, concrete shoulders will minimize overstressing of the edge of concrete pavement, reducing the occurrence of edge spalling and cracking.

(g) In order to minimize longitudinal and transverse cracking, the specifications should be tightened up to insure that all joints are sawcut as soon as the concrete is in a non-plastic state. The longitudinal joint should be cut to a depth of 1/3 the pavement thickness within 12 hours of slipforming.

(h) Experience has shown that all joint sealing must be of high quality and maintained to an equally high caliber. Cracks must be sealed as part of ongoing maintenance to prevent water infiltration to the pavement subbase interface. Stepping has already occurred on some of the longitudinal cracks on Hwy. 3N where sealing operations had been delayed.

(i) Performance results indicate the inclusion of a durable stabilized open graded drainage layer into the rigid pavement section not only improves the overall pavement serviceability but provides a stable, level, and easy to construct platform for placement of the surface pavement. Preliminary results suggest a significant potential for increases in both the pavement life and performance of this lower cost rigid pavement design.

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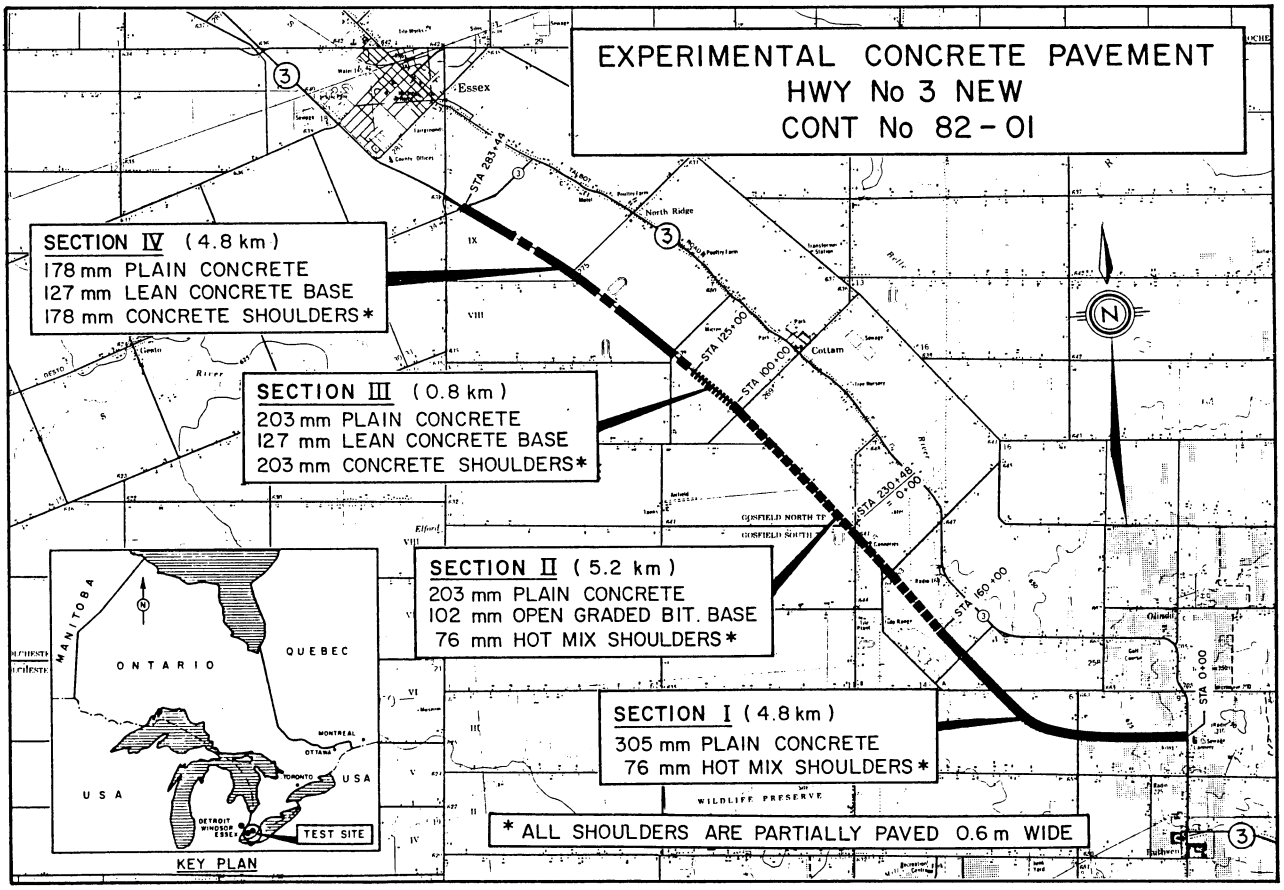
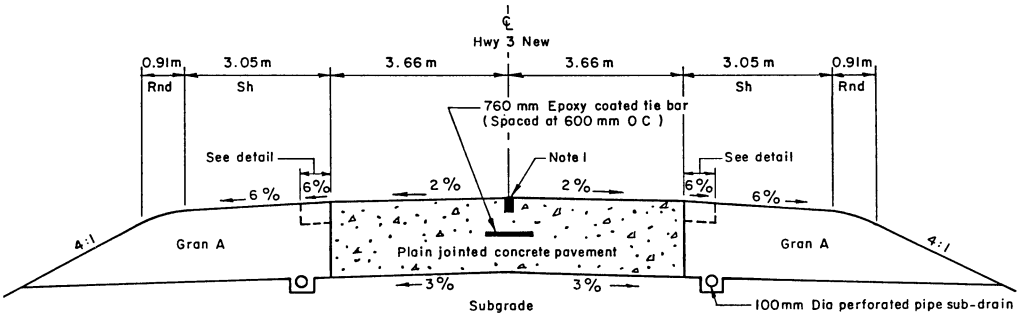
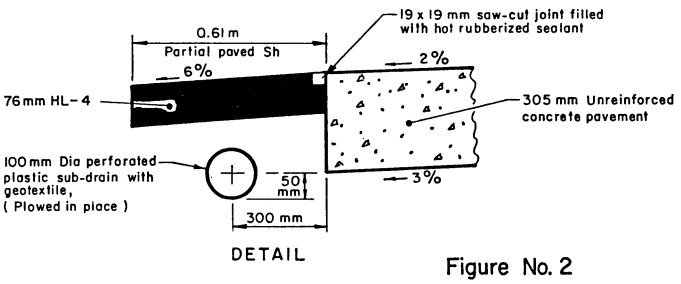


Figure No. 1

CONCRETE PAVEMENT SECTION I

305 mm CONCRETE PAVEMENT ON SUBGRADE





NOTES:

1 ℄ joint saw-cut to 56 x 6 mm and underfilled with hot poured rubberized joint sealing compound.

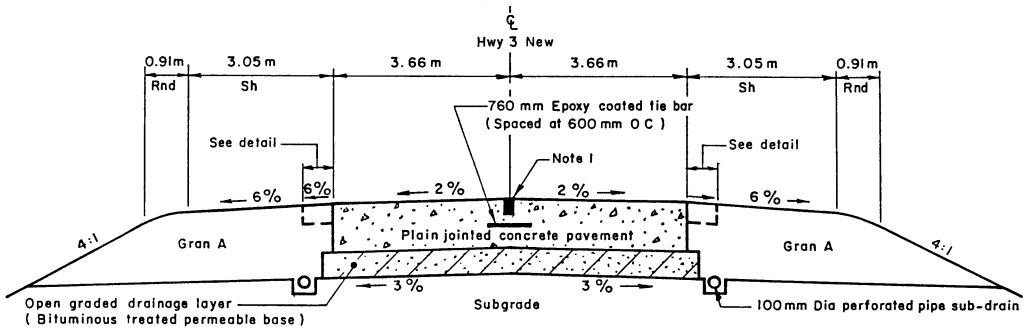
A Conversion:
1 mm = 0.039 inches.
1 m = 3.28 feet

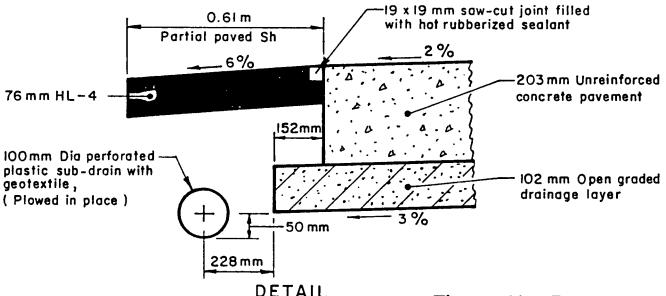
B Not to scale

Figure No. 2

CONCRETE PAVEMENT SECTION II

203 mm CONCRETE PAVEMENT ON 102 mm O.G.D.L.





NOTES:

1 ℄ joint saw-cut to 56 x 6 mm and underfilled with hot poured rubberized joint sealing compound.

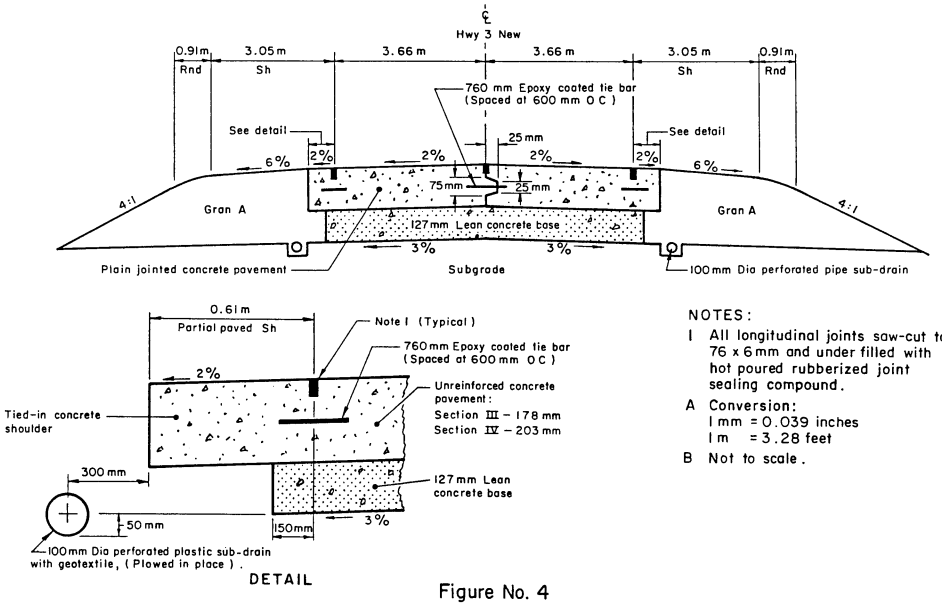
A Conversion:
1 mm = 0.039 inches.
1 m = 3.28 feet

B Not to scale

Figure No. 3

CONCRETE PAVEMENT SECTIONS III AND IV

178 mm AND 203 mm CONCRETE PAVEMENTS ON 127 mm LEAN CONCRETE BASE



NOTES:

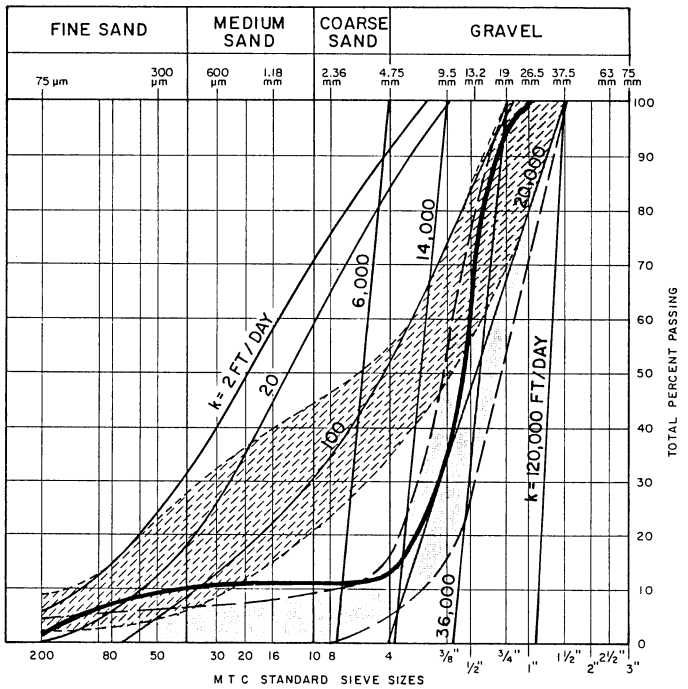
1 All longitudinal joints saw-cut to 76 x 6 mm and under filled with hot poured rubberized joint sealing compound.

A Conversion:
1 mm = 0.039 inches
1 m = 3.28 feet

B Not to scale.

Figure No. 4

OPEN GRADED DRAINAGE LAYER–HWY 3 N PERMEABILITY AND GRADATION PLOT



LEGEND:

Granular 'A' Type I (Base Course)

O G D L Gradation Limits

Actual Hwy 3 New O G D L Mix Gradation

Figure No. 5

LAB TEST RESULTS – HWY 3 NEW, LEAMINGTON

TEST	LEAN CONCRETE BASE						CONCRETE PAVEMENT					
	No of Tests	Mean	Standard Deviation	Coefficient of Variance	Specs Require-ments	% Within Limits	No of Tests	Mean	Standard Deviation	Coefficient of Variance	Specs Require-ments	% Within Limits
	n	$\bar{X}$	S_x	V			n	$\bar{X}$	S_x	V		
Slump (Min)	108	62.9	21.2	33.7	30 - 80	72.9	477	55.4	22.9	41.3	30±10	19.1
Air Content (%)	108	7.17	0.8	11.2	8.0+2.0	92.8	477	5.98	1.00	16.7	6.0±1.5	86.6
28 Day Compression Strength (MPa)	16	15.2	2.67	17.6	10.0 (min)	97.4	67	35.7	4.10	11.5	*	100.0
Core Compression Strength (MPa)	46	18.8	2.31	12.3			119	36.8	3.82	10.4		
Core Length (mm)	46	131.6	14.3	10.9	127±20	81.7						
— 178mm Slab							40	179.2	9.1	5.1	178±12	80.8
— 203mm Slab							44	200.9	16.9	8.4	203±12	51.9
— 305mm Slab							35	305.6	12.9	4.2	305±12	65.1

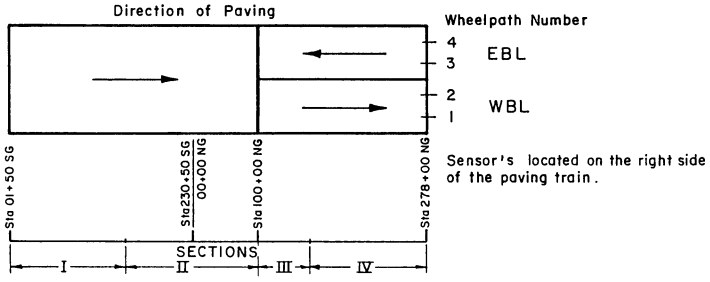
* Average of all groups of three consecutive tests = 25 MPa. Each individual test minimum of 21.5 MPa.

Figure No. 6

SUMMARY OF PROFILE INDEX TEST RESULTS

	Section Traversed					
	1000 ft		4000 ft		One day's paving	
	Number of Wheelpath Averaged *					
	2	4	2	4	2	4
Number of Sections, (n)	36	33	9	8	8	14
Mean Profile Index, inches / mile ($\bar{X}$)	4.78	9.72	4.93	9.75	4.81	9.99
Maximum Profile Index	13.9	19.9	7.7	16.4	7.3	18.1
Minimum Profile Index	0.8	2.4	1.9	5.3	2.7	5.2
Range, inches / mile	13.1	17.5	5.8	11.1	4.6	12.9
Standard Deviation, (S_X), inches / mile	2.88	4.75	2.15	4.13	1.83	3.51
Coefficient of Variation, (V), %	60.2	48.9	43.6	42.4	38.1	35.1

* Two lanes (four wheel paths) where placed with one pass of the paving train from Station 01+50 SG to 230+50 SG and 00+00 NG to 100+00 NG.
One lane (two wheel paths) were placed from Station 100+00 NG to 278+00 NG and 278+00 NG to 124+00 NG.



NOTE:
All measurements taken using California Type Profilograph.

Figure No. 7

PLOT OF AVERAGE ANNUAL ROUGHNESS

HWY 3 NEW WESTBOUND

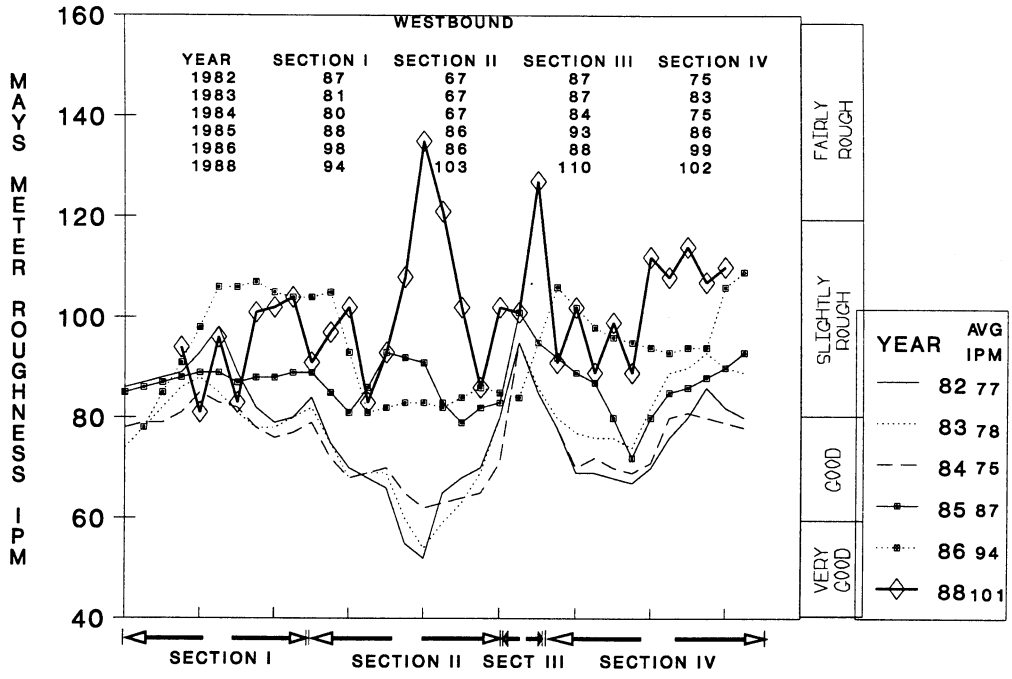


Figure No. 8

PLOT OF AVERAGE ANNUAL ROUGHNESS

HWY 3 NEW EASTBOUND

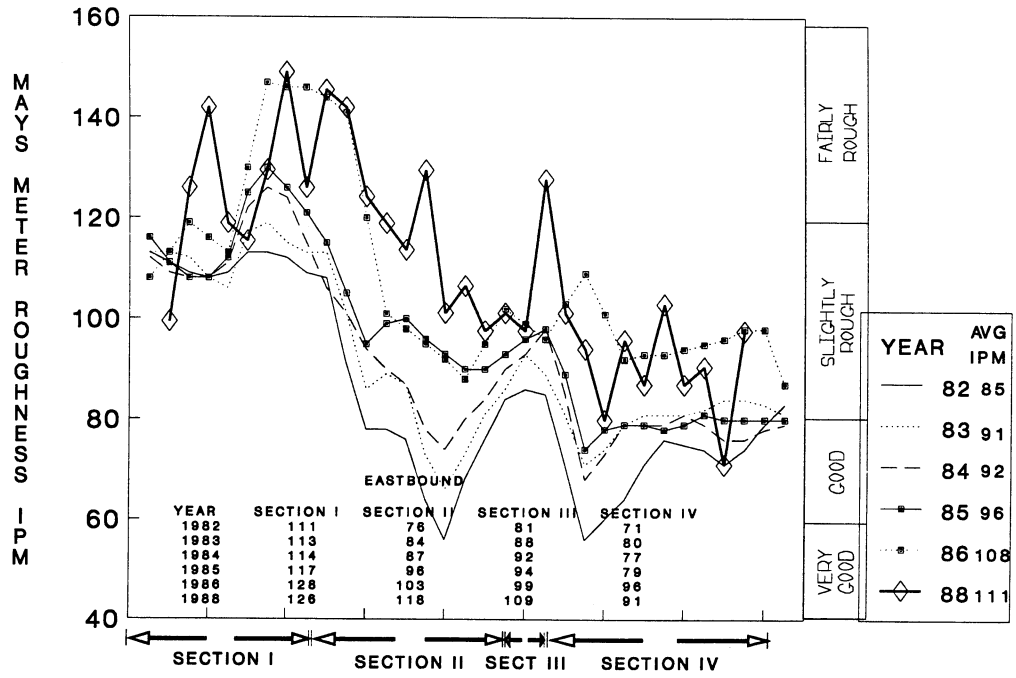


Figure No. 9

SKID RESISTANCE SURVEY
HWY 3 NEW WESTBOUND

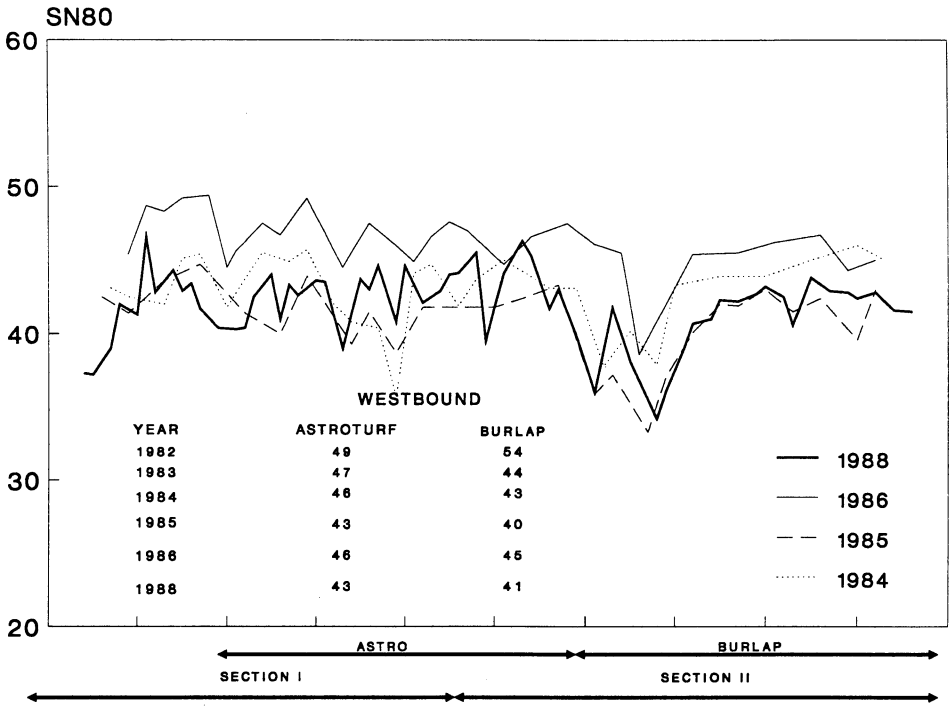


Figure No 10

SKID RESISTANCE SURVEY
HWY 3 NEW EASTBOUND

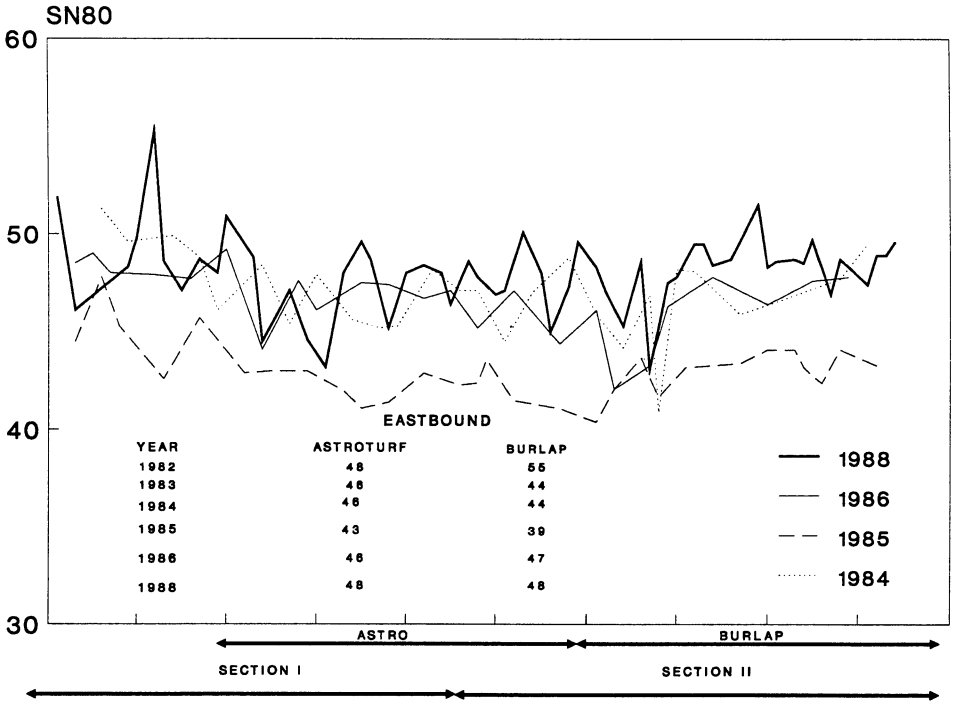


Figure No. 11

SKID RESISTANCE

DATE TESTED	ASTROTURF		BURLAP	
	WEST	EAST	WEST	EAST
OCT 1982	49	48	54	55
SEP 1983	47	46	44	44
NOV 1984	46	46	43	44
OCT 1985	43	43	40	39
JUN 1986	46	47	45	47
AUG 1988	43	48	41	48

SKID RESISTANCE SURVEY
HWY 3N - ASTROTURF AND BURLAP TEXTURES

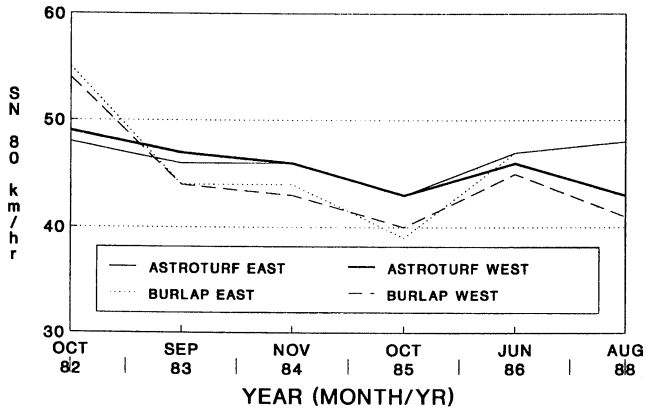
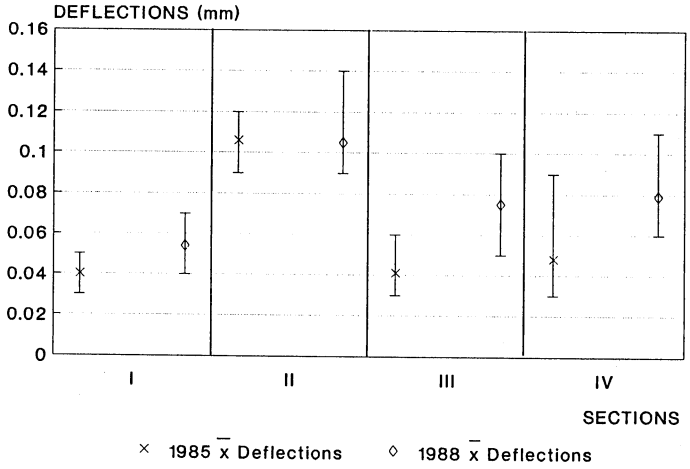


FIGURE 12

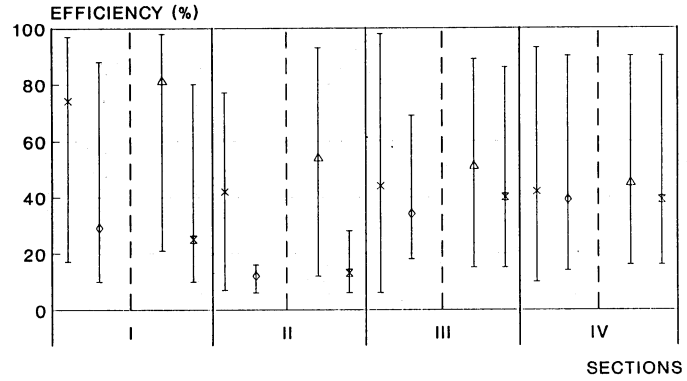
CENTRE SLAB DEFLECTIONS (40KN LOAD)



SECTION NO.	I		II		III		IV	
DATE TESTED	SEPT./85	NOV./88	SEPT./85	NOV./88	SEPT./85	NOV./88	SEPT./85	NOV./88
$\bar{x}$ (mm)	0.04	0.054	0.106	0.105	0.041	0.075	0.048	0.079
Sx (mm)	0.007	0.008	0.011	0.014	0.009	0.016	0.017	0.017
RANGE (mm)	.03-.05	.04-.07	.09-.12	.09-.14	.03-.06	.05-.10	.03-.09	.06-.11

Figure No. 13A

LOAD TRANSFER EFFICIENCY



SECTION NO.	I				II				III				IV			
LOCATION	A.S.		L.S.		A.S.		L.S.		A.S.		L.S.		A.S.		L.S.	
YEAR TESTED	85	88	85	88	85	88	85	88	85	88	85	88	85	88	85	88
x̄ (%)	74	29	81	25	42	12	54	13	44	34	51	40	42	39	45	39
Sx (%)	33	25	32	22	28	7	34	5	39	21	31	21	28	22	32	23
RANGE (%)	17-97	10-88	21-98	10-80	7-77	6-16	12-93	6-28	6-98	18-69	15-89	15-86	10-93	14-90	16-90	16-90

A.S. • Approch Slab
L.S. • Leave Slab

Figure No. 13B

SUMMARY OF ANNUAL DISTRESS SURVEY

PAVEMENT SECTION / DISTRESS TYPE	1983 Survey No of Cracks				1984 Survey No of Cracks				1985 Survey No of Cracks				1988 Survey No of Cracks			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
Transverse Cracking																
- Partial Lane Width	–	–	–	2	–	–	3	14	2	–	3	22	2	3	3	25
- Full Lane Width	5	–	11	74	6	–	18	202	6	3	26	240	6	15	36	244
Longitudinal Cracking	–	–	1	2	–	–	5	25	4	–	6	41	8	–	6	45
Accumulated Length, metres	–	–	15	107	–	–	81	322	26	–	86	505	33	–	89	600
(feet)			*(50)*	*(350)*			*(265)*	*(1055)*	*(85)*		*(280)*	*(1657)*	*(108)*	–	*(292)*	*(1968)*
Corner Cracking	1	4	–	16	2	9	–	24	6	10	–	30	6	14	–	30
Joint Spalling	–	1	–	5	–	4	1	5	3	6	1	7	3	6	1	7
Total No of Cracks per Section	6	5	12	99	8	13	27	270	21	19	36	340	25	38	46	351
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
per (mile)	*(2.0)*	*(1.5)*	*(24.0)*	*(33.0)*	*(2.7)*	*(4. 1)*	*(54.0)*	*(90.0)*	*(7.0)*	*(5.9)*	*(72.0)*	*(113)*	*(8.3)*	*(11.9)*	*(92.0)*	*(117)*

Figure No 14

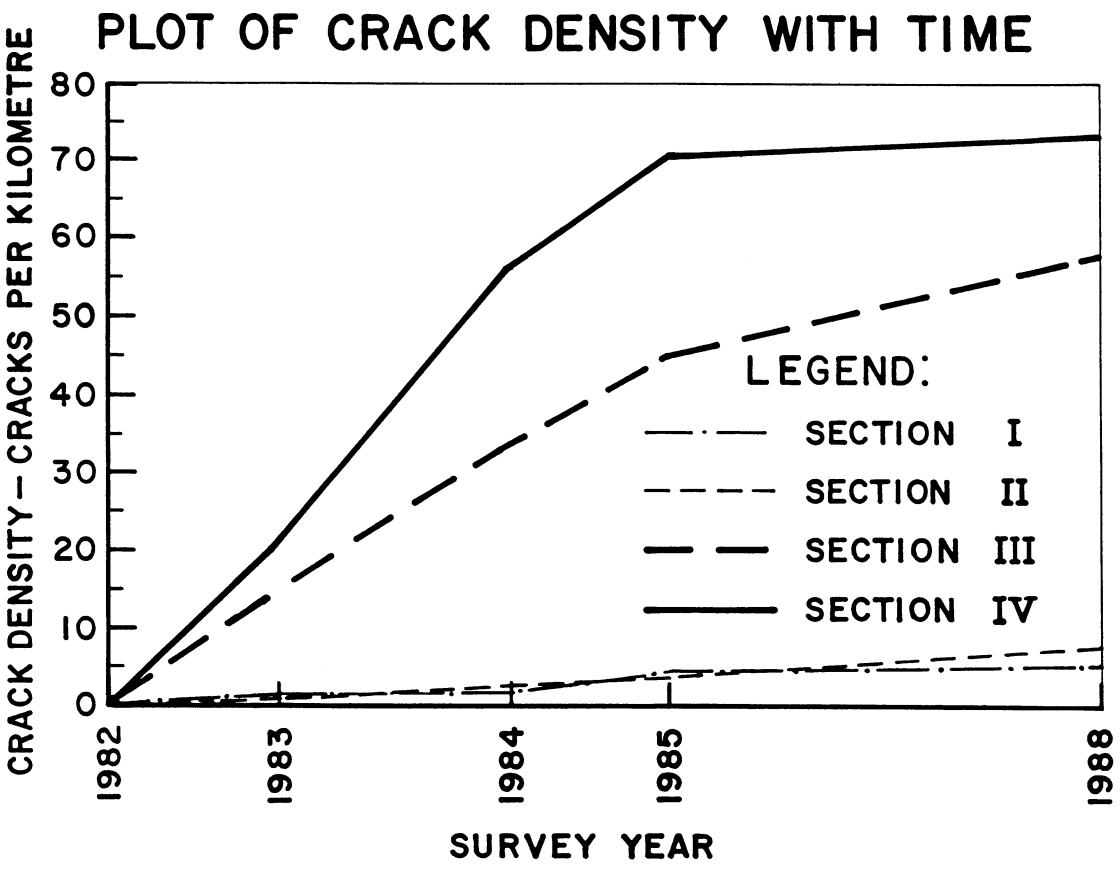


Figure No. 15

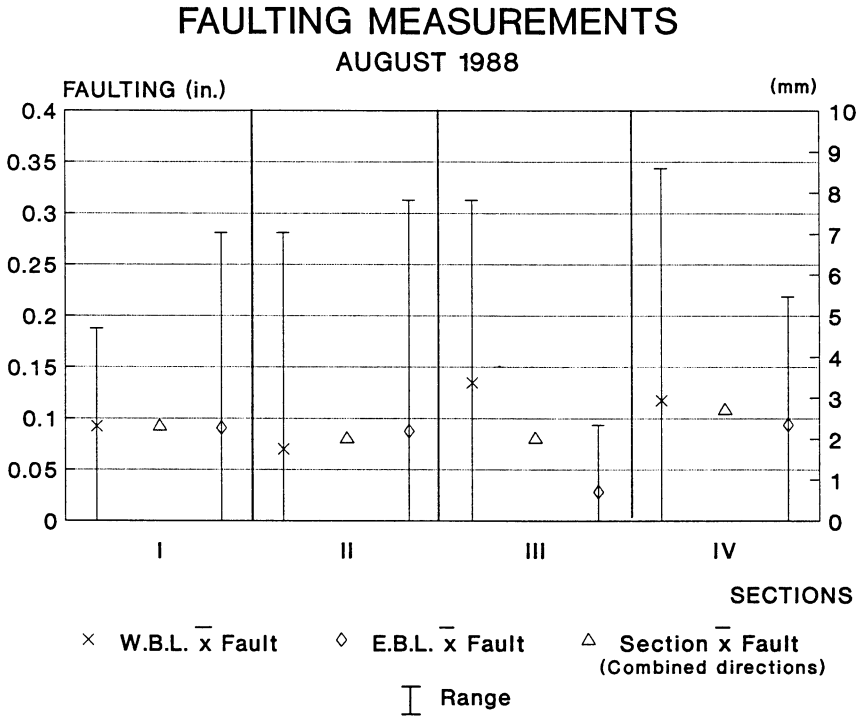


Figure No. 16