

Design, Construction, and Performance of Four Experimental Concrete Pavement Sections in Ontario

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Synopsis: Recent innovative developments in concrete pavement design methodology, material specification, construction techniques, and pavement drainage systems prompted Ontario's Ministry of Transportation and Communications to consider alternative rigid pavement designs.

In order to evaluate the relative performance of several concrete pavement designs, a 15.6 km (9.7 mi) long experimental rigid pavement was constructed in 1982 incorporating four different pavement designs, three shoulder designs and two types of surface textures.

A summary of the design and construction details, plus the initial performance results of an ongoing monitoring program are documented in this paper.

Early performance observations indicate 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.

Continuing performance verification of preliminary conclusions indicate the new designs have resulted in a significant saving in construction materials and costs plus increased durability and performance with a corresponding reduction in future maintenance costs.

Keywords: base courses; concrete construction; concrete pavements; deflection; drainage; load transfer; performance; skid resistance; structural design; surface roughness; test roads

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INTRODUCTION

In 1982, Ontario's Ministry of Transportation and Communications constructed a 15.6 km (9.7 mile) stretch of experimental concrete pavement which incorporated three thicknesses of plain jointed undowelled concrete pavement and two shoulder designs plus two surface textures.

The concrete pavement was constructed as part of Contract 82-01, Hwy. #3 (New) from Ruthven to Essex, refer to site plan Figure 1, and 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 over 102 mm (4 inch) open graded drainage layer (bituminous treated permeable base) with 76 mm (3 inch) thick hot mix partially paved shoulders
- (III) 0.8 kilometres (0.5 miles) of 203 mm (8 inch) thick plain jointed undowelled concrete over 127 mm (5 inch) lean concrete base with 203 mm (8 inch) tied-in plain concrete shoulders
- (IV) 4.8 kilometres (3 miles) of 178 mm (7 inch) thick plain jointed undowelled concrete over 127 mm (5 inch) lean concrete base with 178 mm (7 inch) tied-in plain concrete shoulders

Transverse joints in all sections were skewed and randomly spaced at 4, 5.8, 5.5, 3.7 m (13, 19, 18, 12 ft), 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.

Detailed sections of the four pavement designs are shown on Figures 2, 3, and 4.

The subgrade throughout was a silty clay.

A 100 mm (4 inch) diameter perforated plastic sub-drain wrapped with geotextile was installed 300 mm (12 inch) outside all pavement edges. The pipe invert was located 50 mm (2 inch) into the subgrade and backfilled with Granular 'A' base course materials. Refer to detail sketches on Figures 2 to 4.

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

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

- Pavement Thickness
- Roughness
- Skid Resistance
- Load/Deflection Characteristics
- Pavement Condition Surveys
- Crack Survey
- Joint Movement (horizontal and vertical)
- Subgrade Moisture Content
- Noise Levels
- Traffic Volumes

This paper summarizes these measurements, at the completion of construction and over the next 3 years, presents some observations about the performance of various design features, and points out lessons to be learned from the experiment.

BACKGROUND

Prior to 1968, heavy duty concrete pavements constructed in Ontario were generally 230 mm (9 inch) thick reinforced concrete slabs on 230 mm (9 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

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skewed joints at random spacings of 4, 5.8, 5.5 and 3.7 m (13, 19, 18 and 12 feet) 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 cost makes this design difficult to justify, and escalating construction costs during the ensuing years did not make justification easier. No new concrete pavements were built during the years 1971 to 1983.

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 DETAILS

The experimental pavement was designed as a 7.3 m (24 feet) wide, two lane new facility project with 0.6 m (2 feet) 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 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.

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.

In consideration of the light truck traffic condition, 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.

MATERIALS AND CONSTRUCTION

In general, current M.T.C. 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.

The surface tolerance for earth grades receiving lean concrete or open graded bituminous base was specified as + 12 mm (0.5 inch) from the theoretical grade elevation and cross-section. Prior to laying the second or subsequent base lane width, the subgrade was scarified, reshaped and compacted using an approved 8 ton roller to standard M.T.C. compaction requirements.

The coarse aggregate for the bituminous treated Open Graded Drainage Layer (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.

The O.G.D.L. is basically an open textured, uniformly graded coarse aggregate with 2% A.C. content which was placed in a 100 mm (4 inch) lift directly on the clay subgrade using conventional paving techniques. 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.

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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 2.8×10^{-1} cm/sec (804 ft/day). Based on laboratory permeable testing of cored field samples the average vertical permeability = 1.2×10^{-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.

The 100 mm subdrain 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.

Complete details of the materials and methodology used in construction of the experimental sections are described in Reference 9.

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 feet), 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 ready mix concrete was used for the full depth slab, Section I, to supplement the production of the site plant. Concrete placement commenced on 82-09-08 and continued to completion on 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 Plain Concrete	\$ 220,300 (354,600)
II	5.2 (3.2)	~203 mm P. Concrete 100 mm O.G.D.L. 75 mm Asphalt Shoulder	\$ 218,700 (351,900)
III	0.8 (0.5)	~203 mm P. Concrete 127 mm Lean Concrete Base 203 mm Concrete Shoulder	\$ 272,600 (438,700)
IV	4.8 (3.0)	~178 mm P. 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 Reference 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 reference (7).

Roughness Measurements

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

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This unit comprises a full-sized station wagon with or without a 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. These measurements were completed immediately after construction and subsequently on an annual basis. Tests were carried out in each lane at 80 km/hr. and the average of 3 tests per pavement type are summarized graphically in Figure 8.

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 kph for example.

Each pavement section was tested three times during the first 2 years of service and twice in 1985. Initial measurements were completed immediately after construction and subsequently on an annual basis. Tests were carried out in the left wheelpath of each lane at 80km/hr and the average of 20 tests per surface type are summarized in and plotted on Figure No. 9.

Load Deflection Testing

In order to compare the load/deflection characteristics and the joint load transfer efficiencies of the four pavement sections, a non-destructive load testing program was conducted. For each pavement section, ten slabs were randomly selected for

load 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 versus 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 80km/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 underestimate 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.

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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	Date	Sound Level (dBA)
1	Astroturf Concrete	83 10 06	102.3
1	Burlap Concrete	83 10 06	102.5
1	Old Hot Mix	83 10 06	101.8
1	New Hot Mix	83 10 06	102.1
1	Astroturf Concrete	84 06 13	103.3
1	Burlap Concrete	84 06 13	103.2
1	Old Hot Mix	84 06 13	102.5
1	New Hot Mix	84 06 13	102.7
2	Burlap Concrete	84 06 13	74.8
2	Old Hot Mix	84 06 13	75.3
3	Burlap Concrete	83 10 07	66.3
3	Old Hot Mix	83 10 07	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.

At present, only seasonal variations are apparent in the data, no discussions on trends relating to horizontal joint movement are possible due to the short elapsed time since construction.

Vertical Slab Movements

To monitor vertical slab movements, a series of pins were placed in the outer wheel path at 0.6 metre intervals for approximately 20 metres in all four pavement sections.

Measurements of vertical slab movements have been undertaken at yearly intervals. No discussion of results is presented due to the limited amount of data acquired to date.

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 are being taken many times per year, with preliminary data reflecting seasonal fluctuations but little annual gross change in moisture contents.

Crack Survey

A detailed crack survey, using the methodology outlined in Reference 8, was initiated one year after construction and updated at yearly 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 No. 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

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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. In general, the following 4 year trends are apparent:

- 1) Overall the eastbound lanes have a higher average roughness value (93 IPM) as compared to the westbound lanes (82 IPM), a difference of some 13%.
- 2) The E.B.L. of Section I has the highest average roughness (115 IPM) while the W.B.L. of Section II has the lowest (72 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 (77 vs 81 respectively). This is indicative of the improved surface tolerance achieved by twin grade controls (ie. travelling ski and stringline) used on the E.B.L. single lane width.
- 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 55 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 roughness of the concrete pavement sections can be described as ranging from good to slightly rough which is in agreement with interpretation of the previously mentioned Profile Index results.

Skid Resistance

The results of skid trailer testing on the 4 experimental concrete sections are summarized in Figure No. 9.

The initial magnitude of skid resistance for the burlap drag and astroturf surfaces appear to be comparable with other projects that this Ministry has constructed.

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 where 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 is anticipated that the rate of reduction of skid resistance will be 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 under varying loads and load transfer results are shown on Figure 10.

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

Section IV with only 178 mm of concrete on lean concrete base exhibited larger deflections in the order of 20 to 30% greater when compared to Section I's performance under varying load levels. Of notable interest, is the significantly higher deflections, in the order of 165%, experienced by the concrete

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pavement on O.G.D.L. (Section II) as compared to Section I. From a design point of view, Sections I, III & IV have similar structural stiffness characteristics, however, Section II has significantly less.

The results of load transfer testing are interesting. Although Section I had the highest mean load transfer efficiency (74 to 81%) indicating good load transfer performance, there was still a significant range in the values (17 to 98%). Generally, slab thickness influences load transfer durability for two reasons:

- 1) the aggregate interlock is improved since the contact area is larger.
- 2) the increase in the inertia of the slab reduces the deflections and hence the rate at which the effectiveness of load transfer decreases.

The other three Sections exhibited only fair load transfer performance, with the data being questionable due to their high scatter and variability.

The results reinforce the concept, that under heavy axial loads, relying on aggregate interlock at the joint provides only a marginal load transfer mechanism. Under frequent heavy loading, large deflections at the joint will wear out this aggregate interlock and the pavement will start to fault and become progressively rougher riding.

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 Manifestations

A summary of the extent, number, type, and density of cracking are shown in tabular form on Figure No. 11.

Many of these cracks were observed to occur shortly after construction, indicating improper timing for sawing of the joints.

Briefly, the full depths concrete section and the O.G.D.L. section has undergone negligible amounts of cracking. The former's consisting of slight corner cracking and moderate full lane transverse cracking only on the high side of the super-elevated 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, always associated with joint performance. In addition, both sections show little increase in crack density over the 3 year survey period, as shown graphically on Figure 12.

In Sections with the 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 over the survey period. The longitudinal cracking, very slight to slight, generally runs parallel and some 0.6 to 1.0 metres either side of centreline. Slight 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 Sections III and IV has significantly increased from 15 and 21 in 1983, to 34 and 56 in 1984, and 45 and 71 in 1985. A plot of distress density versus survey date is shown on Figure No. 12. All major cracks noted in the 1983 survey were routed and sealed in early 1984, however, no additional sealing has been carried out until the end of 1985.

The partial asphalt shoulders are depressed and exhibit both transverse and diagonal cracking. At these locations where the transverse joints intersect the pavement edge, a high incidence of slight spalling of the joint (up to 75 mm in length) is observed.

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. Refer to appended photos. Visual examination and analysis of the cores indicate the following:

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- 1) In 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.
- 3) There are no longitudinal cracks at or adjacent to the pavement edge line in the concrete shoulder section where one or both the sawcut longitudinal joint and tie bar were omitted.
- 4) No longitudinal crack was found beneath the typical pavement edge/shoulder joint.

Performance after Three Years of Traffic

The latest pavement condition report, as prepared by the Regional Geotechnical Section, is shown as Figure No 13. An assigned condition rating of 95 is representative of generally good performance, however, the magnitude of cracking in Section III and IV is becoming a concern.

Traffic readings taken two years after completion indicate AADT's ranging from 6,000 to 6,800 for Section III and IV, and from 3,900 to 4,200 for Section I and II. All four sections have a measured commercial traffic count of 10%. These figures are well within the 20 years projected AADT of 8600 and 13% used in design.

CONCLUSION AND RECOMMENDATIONS

This paper summarizes the design, construction, and performance results of three years' observations on four experimental concrete sections. Data collection is ongoing and performance results will be reviewed on a continuing basis.

The following general conclusions, based on results to date, appear warranted:

- a) 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.

b) Both sections I and II are showing a comparable high level of performance and serviceability to date. This is of significant note since the O.G.D.L. design has the lowest design sectional stiffness and the full depth concrete pavement the highest.

c) Based on the immediate structure response to loadings and the resultant measured deflection, the full depth concrete section exhibited the stiffest response while the concrete on the O.G.D.L. section exhibited the least rigid response.

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 0.3 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 nondowelled joints.

e) Initial conclusions indicate the inclusion of a durable free drainage open graded bituminous treated layer into the 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 lower cost rigid pavement design.

f) Additional observations regarding O.G.D.L. design practice suggest:

- i) the O.G.D.L. should extend a minimum of 300mm beyond the concrete pavement edge and the subdrain plowed directly into the O.G.D.L. to insure uninterrupted flow between the two drainage media.
 - ii) the amount of A.C. used as binder for the O.G.D.L. should be reduced to 1.5% or less. The A.C. binder is only required to insure the integrity of the O.G.D.L. under construction traffic, ie. the content should be minimized.
- g) 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.

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h) Future slipform paving of concrete pavement should be specified with dual grade controls. This will improve pavement surface roughness and insure adherence to pavement thickness tolerance requirements.

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

j) The sawn longitudinal joint and tie bar between the concrete pavement and partial width (0.6 metres) concrete shoulder is not necessary and should be eliminated.

k) 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 $\frac{1}{3}$ the pavement thickness within 24 hours of slipforming.

l) 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 on-going 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 have been delayed.

ACKNOWLEDGEMENT

Grateful acknowledgement is made to Messrs. G. Musgrove, M. Stott, D. Woods, F. Marciello, and C. Blaney for their valuable contributions to data acquisition and summation for this report.

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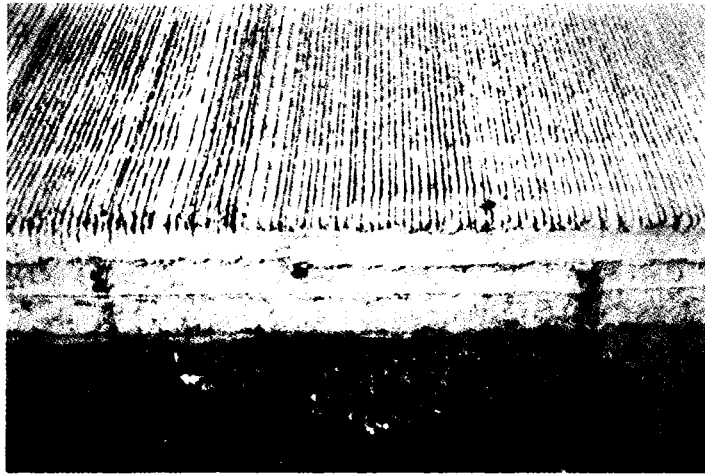


Photo 1—Surface texture of full depth (305 mm) concrete section



Photo 2—Slipformed W.B.L. and shoulder on lean concrete base
(note longitudinal epoxy coated tie bars)



Photo 3—Slipforming E.B.L. and shoulder (sections III & IV)



Photo 4—Paving 100 mm of bituminous treated permeable base (O.G.D.L.)



Photo 5—Open permeable texture of O.G.D.L.

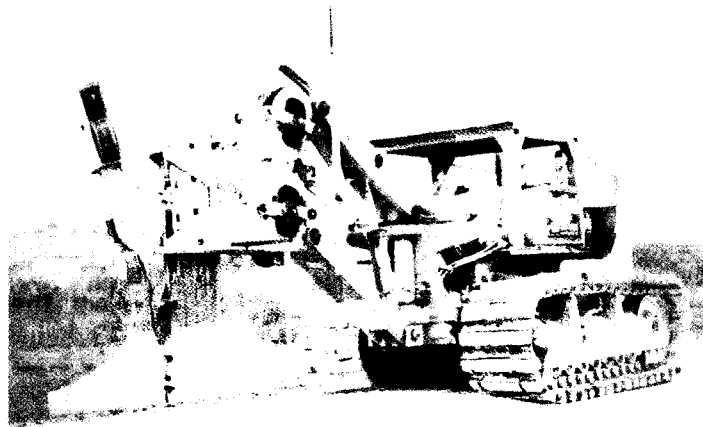


Photo 6—Plow unit used to install 100 mm diameter filter wrapped subdrain

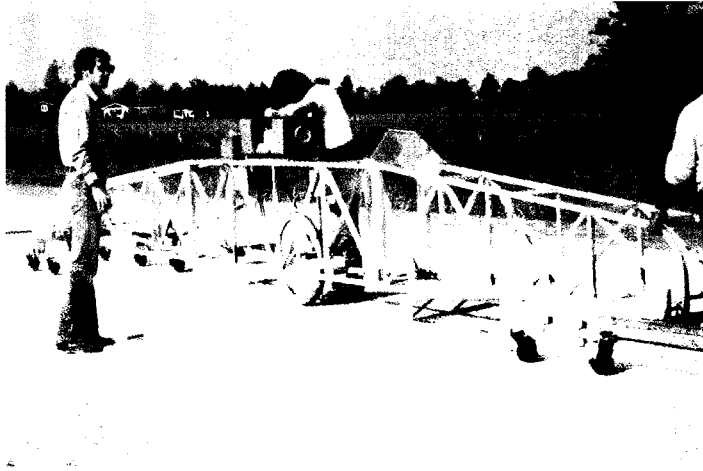


Photo 7—California type profilograph used to measure profile index

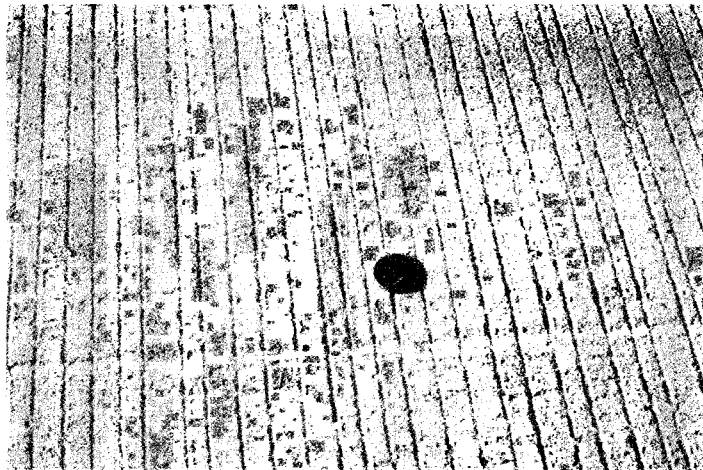


Photo 8—Final surface texture of concrete pavement using Burlap drag

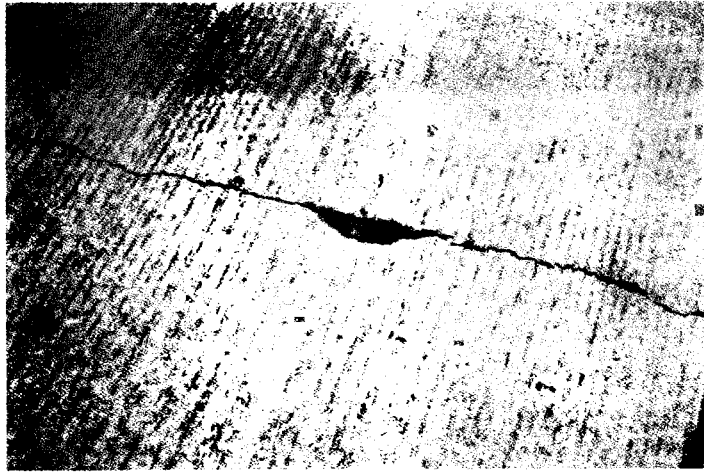


Photo 9—Slight to moderate transverse crack with spall (section IV)



Photo 10—Spalling at intersection of transverse and longitudinal crack (section IV)

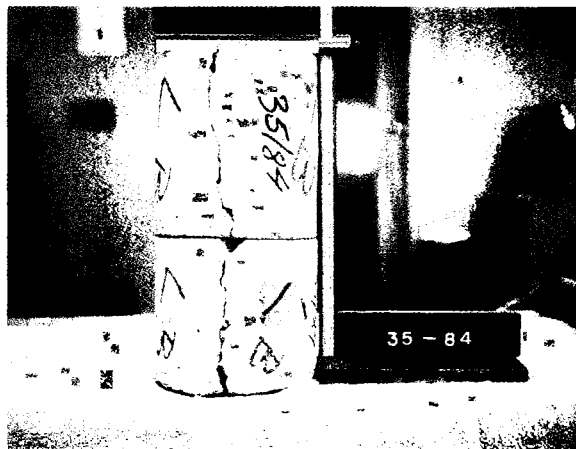


Photo 11—Pavement core showing typical transverse reflection crack from lean concrete base, through 178 mm concrete pavement

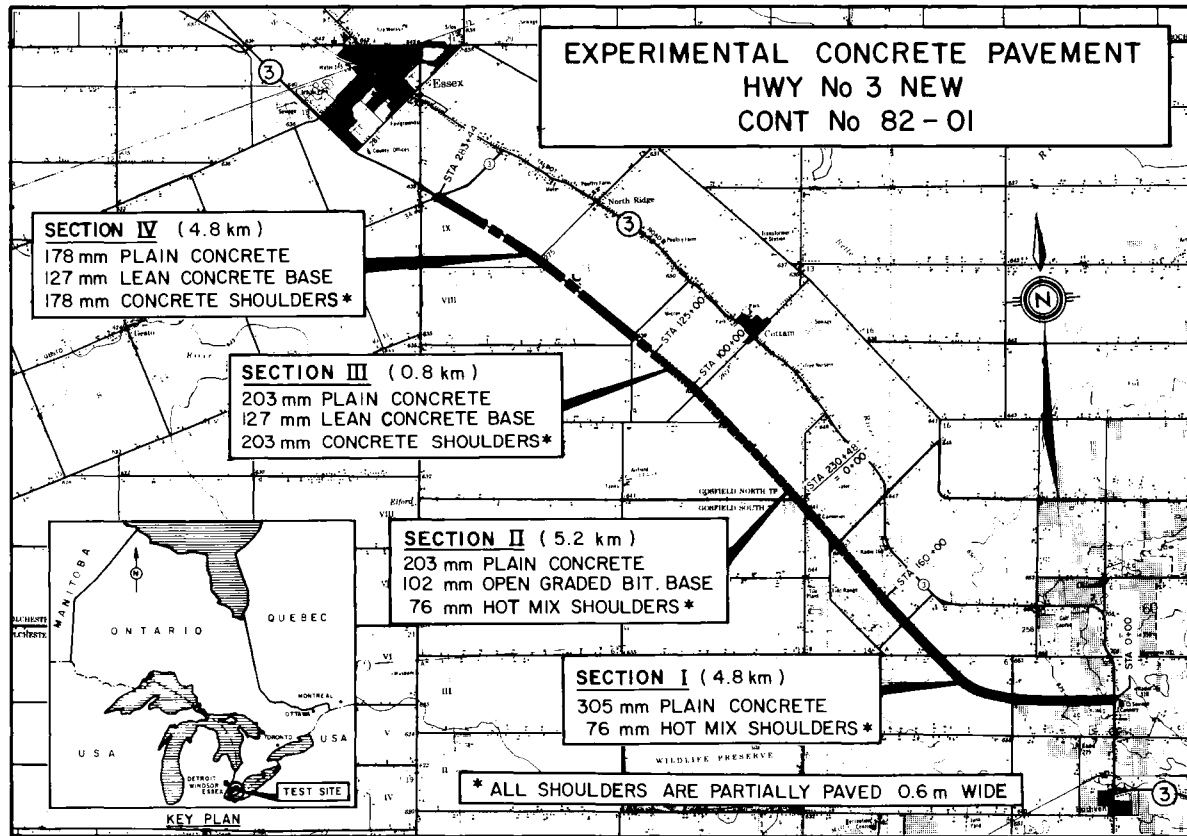
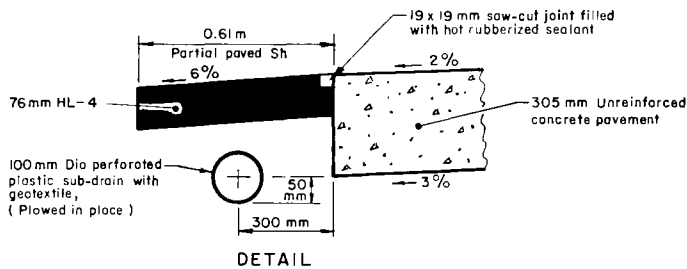
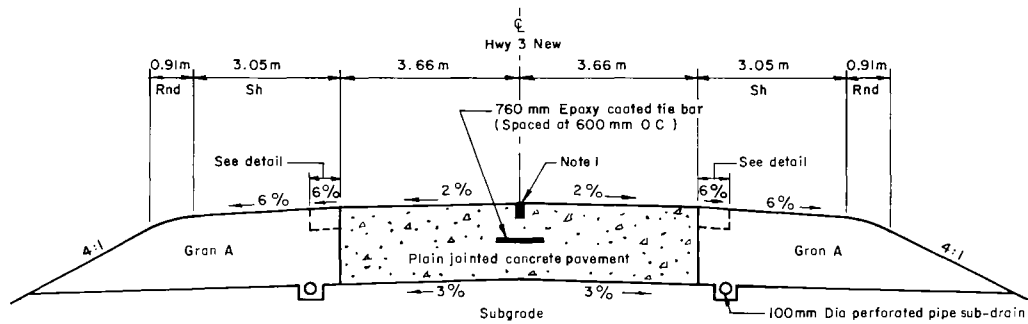


Fig. 1—Key and site plan

CONCRETE PAVEMENT SECTION I

305 mm CONCRETE PAVEMENT ON SUBGRADE



NOTES:

1. C 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

Fig. 2—Concrete pavement section I

CONCRETE PAVEMENT SECTIONS III AND IV

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

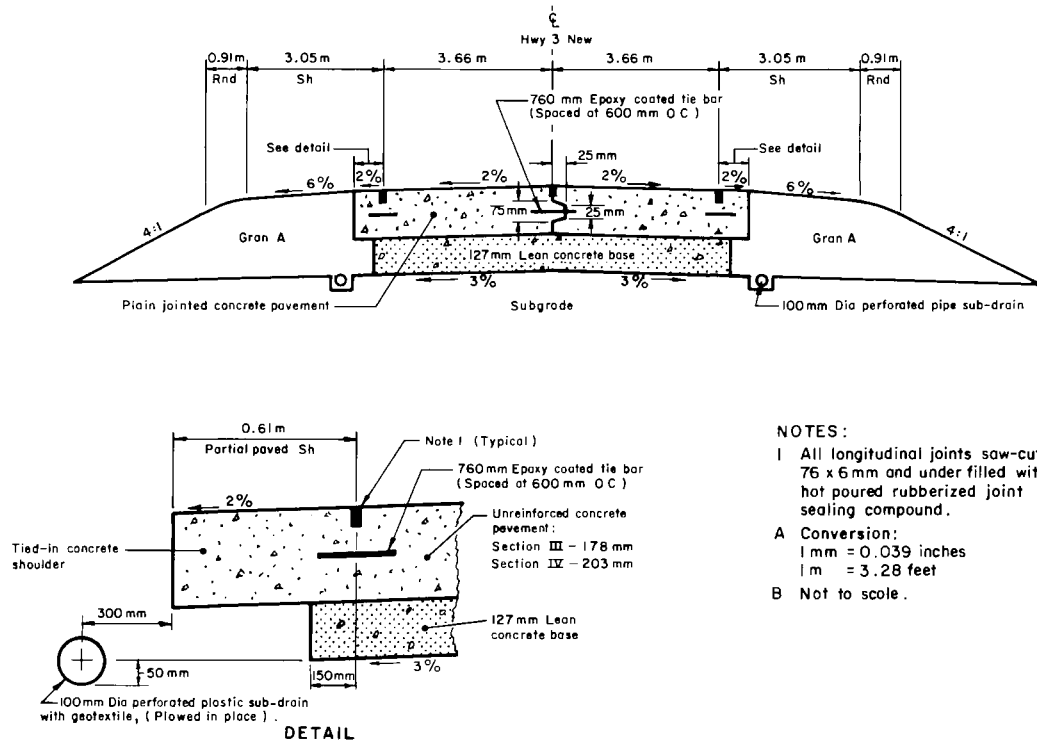


Fig. 4—Concrete pavement sections III and IV

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

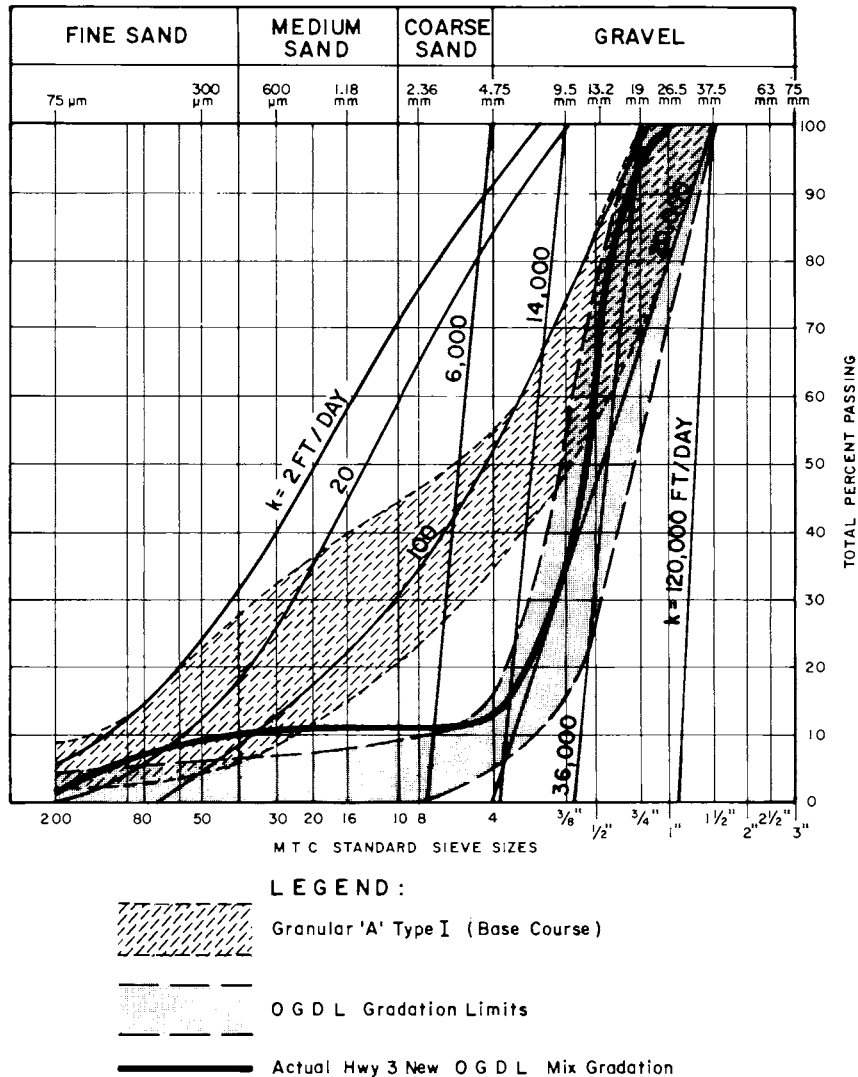


Fig. 5—O.G.D.L., plot of permeability and gradation

LAB TEST RESULTS - HWY 3 NEW, LEAMINGTON

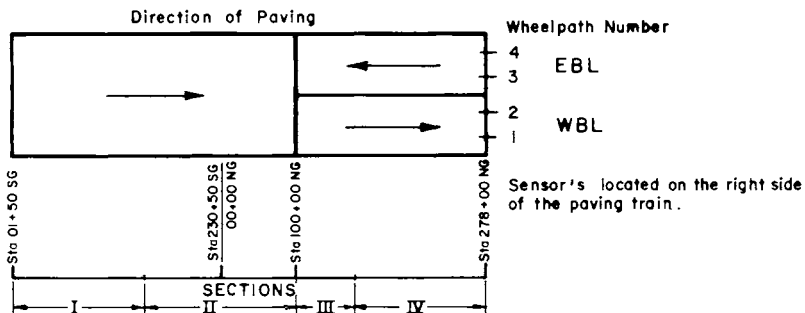
TEST	LEAN CONCRETE BASE						CONCRETE PAVEMENT					
	No of Tests	Mean	Standard Deviation	Coefficient of Variance	Specs Requirements	% Within Limits	No of Tests	Mean	Standard Deviation	Coefficient of Variance	Specs Requirements	% Within Limits
	n	$\bar{X}$	S_x	V			n	$\bar{X}$	S_x	V		
Slump (Min)	108	62.9	21.2	33.7	30 - 80	72.9	477	55.4	22.9	41.3	30 $\pm$ 10	19.1
Air Content (%)	108	7.17	0.8	11.2	8.0 $\pm$ 2.0	92.8	477	5.98	1.00	16.7	6.0 $\pm$ 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 $\pm$ 20	81.7						
- 178mm Slab							40	179.2	9.1	5.1	178 $\pm$ 12	80.8
- 203mm Slab							44	200.9	16.9	8.4	203 $\pm$ 12	51.9
- 305mm Slab							35	305.6	12.9	4.2	305 $\pm$ 12	65.1

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

Fig. 6—Lab test results

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) were 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.

Fig. 7—Profile index test results

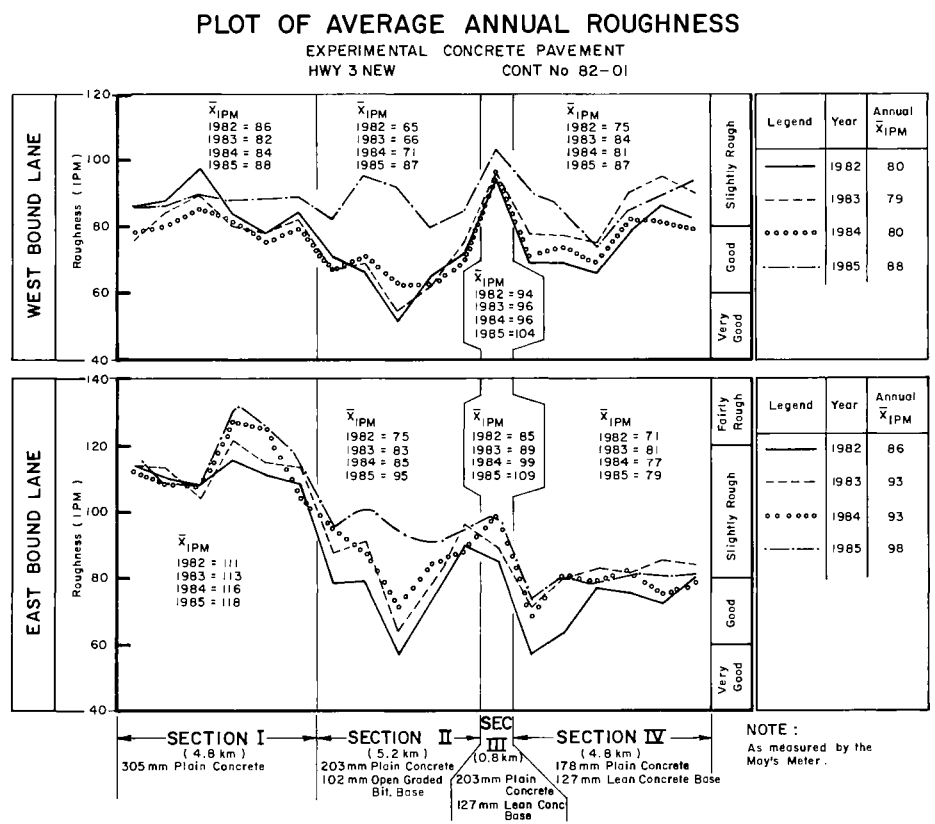


Fig. 8—Plot of average annual roughness

AVERAGE ANNUAL SKID RESISTANCE

CONSTRUCTION TYPE	SURFACE TYPE	LEGEND	SKID RESISTANCE AT 80 km/h			
			Date Surveyed			
			Oct 1982	Sept 1983	Nov 1984	Oct 1985
305mm Plain Concrete	Astro	————	48	46	46	42
203mm Plain Concrete 102mm Open Graded Drainage Layer	Astro	- - - - -	48	46	46	43
	Burlap	- - - - -	55	44	44	40
203mm Plain Concrete 127mm Lean Concrete Base	Burlap	- - - - -	55	44	44	39
178mm Plain Concrete 127mm Lean Concrete Base	Burlap	————	55	44	44	41

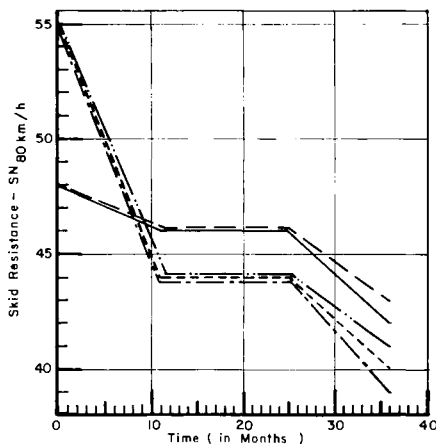


Fig. 9—Average annual skid resistance

CENTRE SLAB DEFLECTIONS

	SECTION							
	I		II		III		IV	
	* 40 kN	** 50 kN	* 40 kN	** 50 kN	* 40 kN	** 50 kN	* 40 kN	** 50 kN
Mean (mm)	0.040	0.058	0.106	0.152	0.041	0.066	0.048	0.075
S_x (mm)	0.007	0.008	0.011	0.014	0.009	0.011	0.017	0.018
Range(mm)	0.03-0.05	0.05-0.07	0.09-0.12	0.13-0.17	0.03-0.06	0.05-0.09	0.03-0.09	0.06-0.12

* 40 kN Simulates 18 kip axial loading.

** 50 kN Simulates 22 kip axial loading.

NOTE:

Loads applied and deflections measured using Dynatest Falling Weight Deflectometer.

LOAD TRANSFER EFFICIENCY

	SECTION							
	I		II		III		IV	
	Approach Slab	Leave Slab	Approach Slab	Leave Slab	Approach Slab	Leave Slab	Approach Slab	Leave Slab
Mean (%)	74	81	42	54	44	51	42	45
S_x (%)	33	31.7	28	34	39	31	28	32
Range (%)	16-98	20-98	7-77	12-93	6-98	9-89	10-93	16-93

Fig. 10—Slab deflections and load transfer

SUMMARY OF ANNUAL CRACK SURVEY

PAVEMENT SECTION DISTRESS TYPE	1983 Survey - No of Cracks				1984 Survey - No of Cracks				1985 Survey - No of Cracks			
	I	II	III	IV	I	II	III	IV	I	II	III	IV
<u>Transverse Cracking</u>												
— Partial Lane Width	—	—	—	2	—	—	3	14	2	—	3	22
— Full Lane Width	5	—	11	74	7	—	18	202	12	3	26	240
<u>Longitudinal Cracking</u>	—	—	I	2	—	—	5	25	4	—	6	41
— Accumulated Length, metres	—	—	15	107	—	—	81	322	26	—	86	505
(feet)			(50)	(350)			(265)	(1055)	(85)		(280)	(1657)
Corner Cracking	1	4	—	16	2	9	—	24	6	10	—	30
Joint Spalling	—	1	—	5	—	4	1	5	3	6	1	7
Total No of Crocks per Section	6	5	12	99	9	13	27	270	27	19	36	340
<u>Distress Density</u>												
— Cracks per kilometre	1.2	1.0	15.0	20.6	1.9	2.5	33.8	56.3	5.6	3.7	45	70.8
per (mile)	(2.0)	(1.5)	(24.0)	(33.0)	(3.0)	(4.1)	(54.0)	(90.0)	(9.0)	(5.9)	(72.0)	(113)

Fig. 11—Summary of annual crack survey

PLOT OF DISTRESS DENSITY WITH TIME

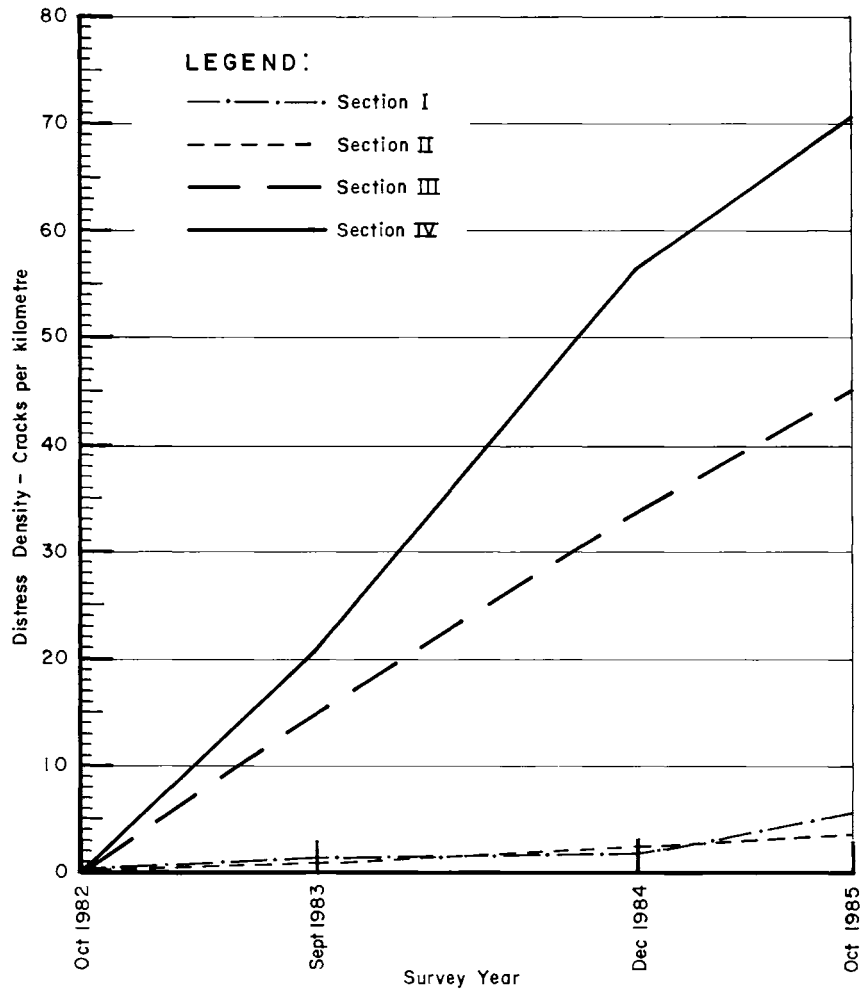


Fig. 12—Plot of distress density with time

38 Pavement Sections



Ministry of
Transportation and
Communications

REGIONAL GEOTECHNICAL PAVEMENT CONDITION REPORT

CONDITION RATING

95

DISTRICT No. 1 HWY. No. 3 W.P. No. _____ LENGTH 15.5 km

LOCATIONS: Highway 3N - from Existing Hwy. 3, 1.6 km N. of Ruthven

W'ly 15.5 km

From L.H.R.S. 12209 Offset 0.0 (Cty. Rd. 34) Last Contract No. 82-01

To L.H.R.S. 12230 Offset 0.0 (Cty. Rd. 23) R.C.R. 8.0
0.5 Part. Paved Shou

PAVEMENT: Type Concrete Width 7.3 m SHOULDER: Width 2.5 Gravel

TRAFFIC: Year ----- A.A.D.T. ----- S.A.D.T. ----- % Trucks -----

SOILS DATA:

This section of highway is located within the Essex clay plain area in which the subsoils are generally silty clay except at the easterly 1 km where the subsoils consist of a layer of fine sand overlying the clays.

PAVEMENT STRUCTURE DATA: This contract 82-01 was new alignment and consisted of the following:

- (i) 300 mm (12") unreinforced concrete with 75 mm P.P.S. (HM) 0+11 to 160+00, S. Gosfield Twp.
- (ii) 200 mm (8") unreinforced concrete with 100 mm G.Bit. Base & 75 mm P.P.S. (HM) 160+00 to 100+00 N. Gosfield Twp.
- (iii) 200 mm (8") unreinforced concrete with 125 mm lean concrete base and 200 mm P.P.S. (Conc.) 125+00 to 283+00 N. Gosfield Twp.
- (iv) 175 mm (7") unreinforced concrete with 125 mm lean concrete base, 175 mm P.P.S. (Conc.) 125+00 to 283+00, N. Gosfield Twp.

All connections and tapers were constructed using HL material plus granular. 0.2 km connection at the north end onto Essex by-pass consisted of 150 mm HM plus 150 mm Granular "A"

MAINTENANCE HISTORY: Contract completed 1983

PERFORMANCE AND CONDITION:

Performance is generally good except some moderate - severe partial width transverse cracking was noted north of Essex Road 27 or in the sections (iii) and (iv), noted above. These cracks intersect with the skewed saw cuts, generally in the driving lane. These cracks have all been sealed using paraplatics.

The cracks occur approximately 150 to 300 m intervals. Very few if any are located in sections (i) and (ii).

A partial paved shoulder was placed throughout with concrete used in Sections (iii) and (iv), while hot mix was used in sections (i) and (ii).

There is considerable drop-off from the concrete paved through lanes to the hot mix partial paved shoulder.

The concrete surface is textured well, using mechanical tines for this purpose.

REMARKS:

Rating - 95. This contract included placing 100 mm plastic perforated subdrains at the edge of pavement.

PROPOSED REMEDIAL MEASURES:

Normal maintenance with any transverse cracks sealed as they develop.

PROGRAM YEAR: Present _____ Suggested _____

DATE OF SURVEY: June 12/84 PREPARED BY: C. M. Bond

Fig. 13—Pavement condition report



Documents

Document Sharing Hub

An Analysis of Routine Maintenance Effects on Rigid Pavements

by T. E. Fwa and K. C. Sinha

Synopsis: A concept of measuring the effects of routine maintenance on rigid pavement performance is presented in this paper. Adopting this proposed concept, a methodology is developed for evaluating such effects based upon pavement performance data and pavement routine maintenance expenditure information. An application of the methodology is presented where the rigid pavements of six Interstate highways in Indiana are analyzed.

Keywords: concrete pavements; costs; evaluation; maintenance; pavements; performance

40 Maintenance Effects

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Kumares C. Sinha is Professor and Head of Transportation Engineering in the School of Civil Engineering at Purdue University. He has had about 25 years of experience in teaching and research in transportation engineering. His current research interest is in the area of highway system management and economics.

INTRODUCTION

It is a well recognized fact that pavement routine maintenance can have significant effects on the performance of a highway pavement. The quality and amount of routine maintenance performed would affect the service life of a pavement. It would also have a direct impact on the total life-cycle cost of the pavement concerned. The ability to predict the effects of pavement routine maintenance therefore is a valuable aid to a highway agency in making sound pavement maintenance planning and funding decisions.

This paper describes a concept for evaluating the effect of pavement routine maintenance on rigid pavement performance. Central to the development of this concept is a hypothesis that there exists a direct relationship between pavement performance and level of routine maintenance performed. Two quantitative parameters are introduced in this paper to represent pavement performance and level of routine maintenance to facilitate the analysis of pavement routine maintenance effects.

An application of the proposed concept is illustrated by means of a case study in which the Interstate highways in Indiana are analyzed. The results of the analysis is expressed in terms of an index, termed as the pavement routine maintenance effectiveness index, which provides a measure of the effects of routine maintenance on pavement performance. Further analysis is performed to correlate these results to variables such as pavement characteristics, traffic loadings, and environmental and climatic conditions. This analysis leads to the development of a regression prediction model for routine maintenance effects on the Interstate rigid pavements in Indiana.

DESCRIPTION OF THE CONCEPT

In an analysis of pavement performance, Fwa and Sinha [1] proposed the use of the parameter PSI-ESAL loss as a representation of pavement deterioration, where PSI is the abbreviation for present serviceability index, and ESAL for equivalent 18-kip single axle load. The definition of PSI-ESAL loss of a pavement is defined in Figure 1 which is obtained by plotting PSI against cumulative ESAL.

Consider a hypothetical case in which the performance curves of three pavement sections of the same structural design are compared. If all the three sections are exposed to identical environmental and climatic conditions and traffic loadings, it is likely that the pavement section with the best level of pavement routine maintenance would have the lowest PSI-ESAL loss, and the pavement section with the lowest maintenance level would have the highest PSI-ESAL loss. This relationship is schematically represented in Figure 2.

The total pavement routine maintenance expenditure for each pavement section is represented by S_1 in Figure 3. Pavement routine maintenance refers to those pavement repair activities that are performed routinely by a highway agency's own force. For pavement sections maintained by the repair crews of a given highway agency with the same technology, it is logical to say that a higher level of routine maintenance would result with higher maintenance expenditure. This is the relationship presented in Figure 3.

The concept presented in Figures 2 and 3 can be extended theoretically to include n levels of routine maintenance expenditures, $S_1, S_2, \dots, S_n$, for which

$$S_1 > S_2 > \dots > S_n \quad (1)$$

Let D_i represent the corresponding PSI-ESAL loss for the performance curve with routine maintenance expenditure S_i , where $1 < i < n$, the concept described implies the following relationship,

$$D_1 < D_2 < \dots < D_n \quad (2)$$

Consider two successive performance curves with maintenance expenditure S_j and S_{j+1} , where $1 < j < (n-1)$ and $S_{j+1} = S_j + \delta S_j$. As the maintenance expenditure difference δS_j between the two expenditure levels becomes infinitely small, the corresponding reduction in PSI-ESAL loss represents the rate of improvement in

42 Maintenance Effects

pavement performance attainable when another unit of expenditure is added to the existing amount S_i . This rate of change in pavement performance expressed as reduction in PSI-ESAL loss per unit expenditure added is known in the present paper as the pavement routine maintenance effectiveness index. Mathematically, we may define

$$M_i = \frac{dD}{dS} \bigg|_{S_i} \quad (3)$$

where M_i is the pavement routine maintenance effectiveness index evaluated at expenditure level S_i . A negative sign is added in Equation (3) so that a positive index value refers to an improvement in pavement performance (due to a reduction in PSI-ESAL loss) as a result of increasing routine maintenance expenditures.

The effectiveness index M provides a convenient measure to compare the relative effectiveness of different maintenance policies. A low index M value infers that there is little improvement in pavement performance for each maintenance dollar spent, while a high index M value implies effective routine maintenance work since each unit maintenance expenditure could produce a large improvement in pavement performance.

AN EXAMPLE OF APPLICATION

An application of the concept described is provided in this section where the rigid pavements of six Interstate highways in Indiana are analyzed. Figure 4 shows the geographical locations of these Interstate highways. The data requirements and the steps involved in the computations are described in the sub-section below, followed by the presentation of the results and statistical analyses of these results.

Data Requirements

Two major categories of data were needed for evaluation of the routine maintenance effects on the Interstate highways. These were the pavement performance data and the pavement routine maintenance expenditure data.

Each highway route was first divided into a number of pavement sections in accordance with the recorded units of pavement performance and maintenance expenditure data. The pavement performance curve of each pavement section was derived from its PSI history. This PSI history was in turn derived from the annual Roadmeter roughness record maintained by the Indiana Department of Highways (IDOH) by means of the following

PSI-Roughness number correlation model [2]:

$$\Psi = 11.73 - 2.83369 * \log (RN) \quad R^2 = 0.68 \quad (4)$$

where Ψ = present serviceability index

RN = Roadmeter counts per mile

R^2 = statistical coefficient of multiple determination

Traffic data were required to compute the cumulative ESAL for establishing the performance curve of each pavement section. These data included traffic volume, traffic stream composition, and vehicle operating weights and axle configuration information. The traffic volume on each pavement section was computed from the AADT (average annual daily traffic) on the traffic maps published annually by the IDOH. The remaining data necessary for ESAL computation were obtained from the records of the 1984 Indiana Highway Cost Allocation Study [3].

The pavement routine maintenance expenditure data used in the present analysis were extracted from the maintenance data base compiled for Indiana highways by the Joint Highway Research Project (JHRP) at Purdue University [4]. An average annual aggregated pavement routine maintenance expenditure was obtained for each pavement section. This expenditure was then divided by the total lane-miles of the pavement section to arrive at a per lane-mile value of the expenditure.

Computational Procedure

The steps involved in the computation of the routine maintenance effects on each of the Interstate highway routes are summarized below:

- [1] Develop the performance curve for each pavement section of the highway route and compute the corresponding PSI-ESAL loss.
- [2] Calculate the per lane-mile pavement routine maintenance expenditure on each pavement section.
- [3] Plot the PSI-ESAL losses derived in step (1) against the corresponding maintenance expenditures computed in step (2).
- [4] Fit a least-square straight line to the data points in step (3) and measure the slope of the fitted line.

Referring to the earlier described concept which was expressed mathematically by Equation (3), the slope measured in step (4), with a change in sign, gives the

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pavement routine maintenance effectiveness index M for the routine maintenance work performed on the highway route. An example of such a plot is shown in Figure 5. Equation (3) indicates that the index M value could vary at different maintenance expenditure values. The straight line fitting in step (4) therefore provides only an average M value for the entire expenditure range considered. Further research is being conducted to investigate the variation of effectiveness index M with maintenance expenditure levels.

Results of Computation

The values of effectiveness index M for the six Interstate highway routes are shown in Table 1. These index M values were computed at a cumulative ESAL level equal to 1.5×10^7 . It is important to mention that the order of magnitude of the effectiveness index M would vary according to the magnitude of cumulative ESAL at which M is evaluated. This means that when the magnitude of an effectiveness index M is mentioned, it is meaningful only if the associated cumulative ESAL level is specified.

The results in Table 1 indicate that there was some variation in the effectiveness of the routine maintenance work performed on different highway routes. Findings of several earlier studies [4,5,6] suggest that the routine maintenance practices in different parts of Indiana were relatively uniform. It is therefore likely that other factors such as intensity and level of traffic loadings, pavement characteristics, and environmental and climatic conditions might be responsible for the variation in the magnitudes of the index M in Table 1. The statistical analyses presented in the following sub-sections help to provide some useful information in this aspect.

Factors Influencing Effects of Routine Maintenance

The factors that were likely to have an influence upon the effects of routine maintenance are identified below: (a) pavement characteristics -- pavement thickness and pavement age; (b) traffic loadings -- intensity and level of loadings; (c) environment and climate -- freezing index, snowfall, rainfall, subgrade moisture, temperature and subgrade soil.

The specific variable used to represent the factors listed above are given in Tables 2 and 3. The pavement characteristics and traffic data were, as described earlier, derived from records maintained by the IDOH, and the JHRP at Purdue University. The regional distributions of the environmental and

climatic variables identified in Table 3 were available from the Indiana climatological data kept by the Agricultural Experimental Station of Purdue University. It should be noted that the freeze-thaw cycle index used in the present study refers to the number of times air temperature crosses the freezing temperature of 32°F .

Each of the variable values in Table 2 and 3 represents a weighted average variable value on a given highway route. These weighted average values were computed by the following procedure. For each pavement section of a highway route, a value of the particular variable was first assigned. The assigned values were then weighted by the lengths of their respective pavement sections to arrive at the weighted average variable value for the highway route.

Correlation Analysis on Routine Maintenance Effects

Presented in this section are the results of statistical correlation analyses which were performed to examine the correlation relationships between the routine maintenance effects on the Interstate highways in Indiana and the variables identified in the preceding section. The pavement routine maintenance effectiveness index was used as a quantitative representation of routine maintenance effects on pavement performance in these analyses.

The statistical correlation matrix is shown in Table 4. At a confidence level of 90 percent, X2, X4 and X12 were each found to have a significant correlation relationship with the effectiveness index M. The best correlation was obtained with variable X4, the intensity of traffic loadings expressed in terms of mean annual ESAL. The positive sign of the correlation coefficient indicates that the effect of routine maintenance became more pronounced as the intensity of traffic loadings increased.

Pavement age, X2, had a negative correlation coefficient with routine maintenance effects. It implies that routine maintenance was more effective in improving pavement performance when carried out at a younger pavement age. This finding appears to affirm the benefits of early preventive maintenance. The positive correlation between subgrade soil reaction, X12, and effectiveness index M simply means that to achieve the same amount of improvement in pavement performance, more routine maintenance expenditure was needed on a pavement with poorer subgrade soil.

The results in Table 4 also indicate that there were comparatively low correlation between

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maintenance effects and climatic variables, X6 through X11. None of these correlation relationships were significant at a confidence level of 90 percent.

Prediction Model of Routine Maintenance Effects

Statistical regression prediction models for routine maintenance effects are developed in this section for the rigid pavements of the Interstate highways in Indiana. The objective is to illustrate that prediction of routine maintenance effects can be made based upon information which are readily assessable by most state highway agencies.

The regression analyses were performed with effectiveness index M as the dependent variable and the variables identified in Tables 2 and 3 as independent variables. Interaction terms which have interpretable physical meanings were also included in analyses. These terms included interaction between traffic and climatic variables, pavement and traffic variables, and climatic and pavement variables.

Several regression models were found to be significant at a confidence level of 95 percent. These models are:

$$M = 7316.40 + 0.0239(X4) - 0.001341(X5) \quad (5)$$

$$M = 18749.379 - 773.717(X2) + 0.003953(X4) \quad (6)$$

$$M = 15428.965 - 840.834(X2) + 40.1118(X12) \quad (7)$$

where all variables are as defined in Table 4.

The statistical characteristics of these models are shown in Table 5. The variable X5 represents the interaction between pavement age (X2) and mean annual traffic loading (X4). Due to the small number of cases available, an adjusted coefficient of multiple determination ($\text{adj. } R^2$) is included for each model to show the effect of this limitation [7].

Comments on Application

It is important to note the results of the correlation and regression analyses provide only statistical relationships between routine maintenance effects and the variables considered. These relationships are not mechanistic in nature. Caution must be exercised in interpreting the meaning of a regression model or the results of correlation analyses by making reference to the inference space of the statistical analyses.

All the six rigid pavement routes had the same slab thickness of 10 inches. The results of the statistical analyses were therefore valid for one slab thickness only. It is likely that if more than one slab thickness were considered, different regression models might be obtained. Similarly, if rigid pavements in several regions with drastically different climatic conditions were analyzed, it is quite possible that climate would show more impacts than was found in the present example.

Although only six rigid pavement routes were included in the example, it nevertheless achieves the following two objectives: (a) it provides a full-scale practical application of the proposed methodology; and (b) it illustrates how additional useful information could be obtained by further statistical analyses.

CONCLUSIONS

A concept of measuring the effects of routine maintenance on rigid pavement performance is presented in this paper. Adopting this proposed concept, a methodology is developed for evaluating such effects based upon pavement performance data and pavement routine maintenance expenditure information. The computation involved is simple. It can be easily programmed and executed on a microcomputer.

The example application presented illustrates the practical feasibility of the methodology. At project level, the analysis provides a useful tool for evaluating the effects of routine maintenance on the pavement considered. At network level, additional analyses may be conducted to examine the regional variation of routine maintenance effects, and how these effects are influenced by pavement characteristics, traffic loadings and climatic conditions.

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Table 1 Pavement Routine Maintenance Effectiveness
Indices for Rigid Pavements in Indiana

Case	Highway Route	Effectiveness Index M ($\times 10^4$)
1	I-94	1.65
2	I-65	1.43
3	I-69	0.53
4	I-70	0.96
5	I-74	0.70
6	I-64	0.97

Note : 1. M is in PSI-ESAL loss/dollar/year/lane-mile .
2. All M values were evaluated at cumulative ESAL
value of 15,000,000 .

Table 2 Pavement and Traffic Characteristics Data

Case	Highway Route	Pavement Age (yr.)	Thickness (in.)	Mean AADT	Cumulative ESAL
1	I-94	14.84	10.00	50,099	32,870,000
2	I-65	13.43	10.00	26,853	22,770,000
3	I-69	19.10	10.00	17,643	14,900,000
4	I-70	16.17	10.00	27,989	20,010,000
5	I-74	20.18	10.00	10,516	9,380,000
6	I-64	13.44	10.00	10,636	6,320,000

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Table 3 Environmental and Climatic Data

Case	Highway	X6	X7	X8	X9	X10	X11	X12
1	I-94	335.7	42.2	36.3	40.0	110.2	50.8	340.1
2	I-65	192.5	30.5	39.6	40.9	111.4	52.1	200.3
3	I-69	212.5	31.8	38.1	40.3	113.7	51.9	179.9
4	I-70	50.0	22.3	40.2	45.4	119.8	53.4	179.9
5	I-74	60.0	22.7	40.0	45.1	117.4	53.3	190.2
6	I-64	0.0	14.4	43.9	52.8	112.1	56.2	195.0

Note: X6 = Freezing index in °F-day
X7 = snowfall in inches
X8 = rainfall in inches
X9 = Thornthwaite moisture index
X10 = freeze-thaw cycle index
X11 = annual average temperature
X12 = soil subgrade reaction in pound per cubic inch