LONG-TERM PERFORMANCE OF AN EXPERIMENTAL COMPOSITE PAVEMENT

J. Ryell and J. T. Corkill, Ministry of Transportation and Communications, Ontario

The report presents an evaluation of an experimental pavement constructed on Highway 401 near Milton, Ontario, in 1959. The experiment consisted of 7 sections of composite pavement and a section of continuously reinforced concrete pavement. The control section is normal reinforced concrete pavement (1959 design). Data on transverse and longitudinal cracking, road ridability, and skid resistance are presented; each pavement section is rated on present performance and economic aspects. Conclusions are that (a) the most satisfactory pavement section in the experiment consists of a $3\frac{1}{4}$-in. bituminous overlay on an 8-in. unreinforced concrete base, (b) mesh reinforcement in the concrete base has no beneficial effect on the overall performance of the composite pavement, and (c) the best composite pavement in this experiment performed significantly better than many comparable rigid and flexible pavements that are located elsewhere on Highway 401 and are of approximately the same age and subjected to a similar traffic volume.

IN THE FALL of 1959, an experimental composite pavement was constructed in southern Ontario as part of a major 4-lane, controlled-access highway. The main objective of the project was to determine whether a pavement combining the load-bearing properties of a concrete base with the smooth-riding properties of an asphalt wearing course could be economically built.

Three basic composite pavement designs were constructed: unreinforced concrete base with an asphalt overlay, continuously reinforced concrete base with an asphalt overlay, and jointed reinforced concrete base with an asphalt overlay.

The 3 pavement types were further varied by the thickness of pavement layers and the weights of reinforcing mesh to give a total of 7 experimental sections (Fig. 1). Those sections were compared in performance to a section of continuously reinforced concrete pavement (0.38 percent longitudinal steel) corresponding to one of the concrete base sections and to a section of normal reinforced concrete pavement (1959 design) constructed immediately west of the experiment.

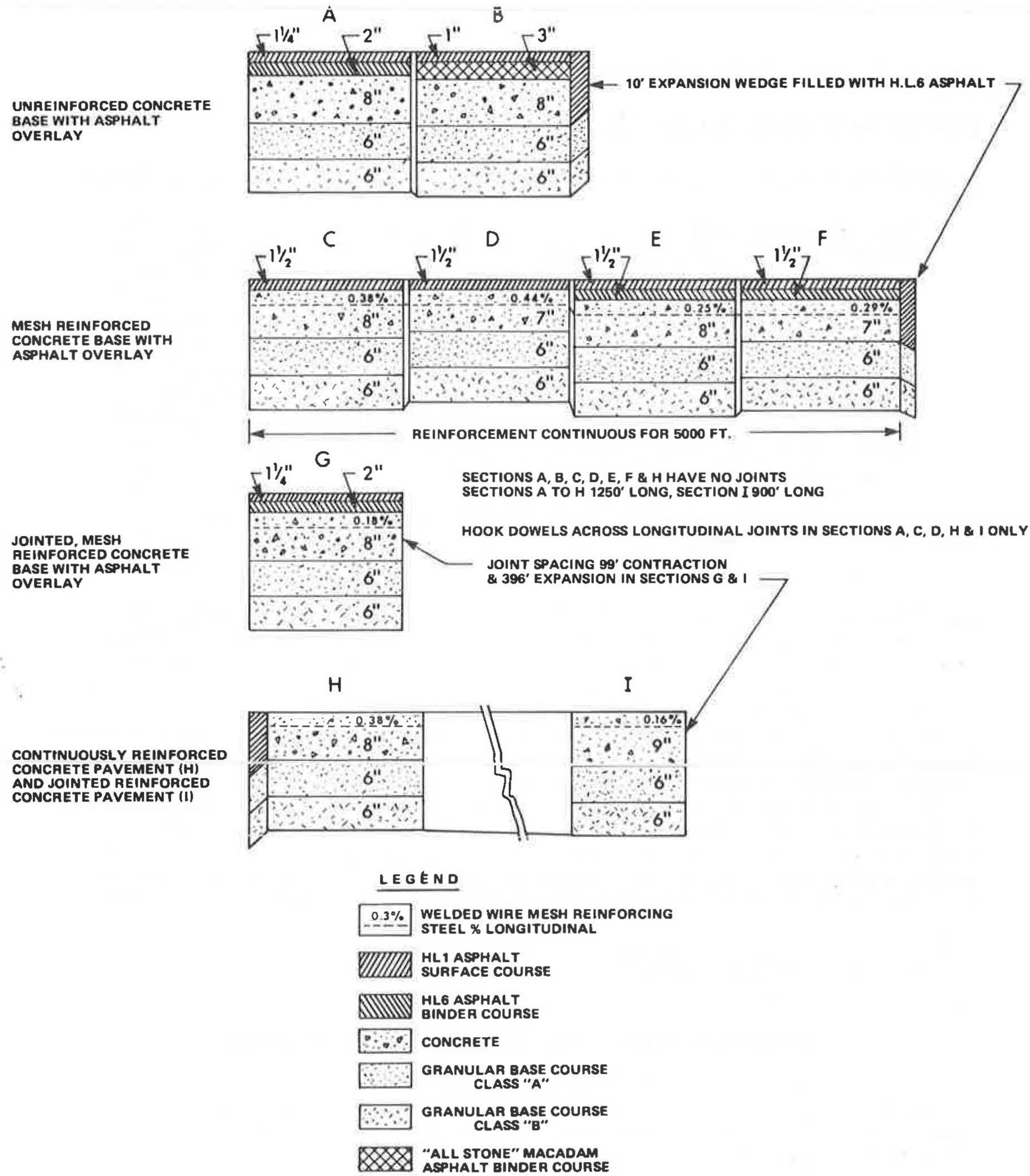
Details of the experimental pavements together with performances during the first 3 years were previously reported (1). This report describes the performances of the pavements after almost 13 years of service.

PRINCIPAL DESIGN AND CONSTRUCTION FEATURES

The important design and construction features of the experiment and the 1972 measurements of the bearing capacity of the granular base course and clay subgrade are summarized below. The subgrade and granular base course were essentially constant throughout the 9 pavement sections; the variables were confined to the concrete and asphalt portions of the pavement structure. Materials used are given in Table 1.

Sponsored by Committee on Design of Composite Pavements and Structural Overlays.

Figure 1. Design features of pavement sections.



Concrete

The concrete materials and mix proportions were similar in all 9 pavement sections and in general are typical of those used in concrete pavement construction in Ontario. The coarse aggregate was crushed dolomitic limestone; the fine aggregate was natural sand. The air-entrained concrete contained a calcium lignosulfonate water-reducing admixture.

Asphalt Surface Course

HL1 asphalt was used as the surface course on 7 of the sections and is typical of the asphalt mixes specified for all major heavily trafficked highways in Ontario. The coarse aggregate in this mix was crushed traprock, a very hard, fine-grained basaltic quarried material. The fine aggregate was natural sand.

Asphalt Binder Course

HL6 asphalt was used as the binder course on sections A, E, F, and G; it is widely used in Ontario as a binder or base course. The coarse aggregate was crushed dolomitic limestone, and the fine aggregate was natural sand.

Macadam Asphalt Binder Course

The "all-stone" macadam asphalt binder course was a relatively new type of bituminous mix in Ontario and consisted of coarse aggregate (crushed dolomitic limestone) and asphalt cement; no fine aggregate was used. It was originally specified to contain a $2\frac{1}{2}$-in. maximum size of coarse aggregate, but the mix proportions and aggregate size were modified in the field because of problems in plant mixing and segregation during laying. The modified asphalt was mixed and laid without further problems.

Design Features

Sections A and B are unreinforced 8-in. concrete slabs with relatively thick asphalt overlays. The 4-in. thickness of HL1 and macadam asphalt in section B was designed to reduce the amount of cracking reflected through the asphalt from the concrete base. Sections C, D, E, and F consist of 7- or 8-in. reinforced concrete base with either 1 or 2 layers of $1\frac{1}{2}$-in. asphalt overlay. Sections C and D contain approximately twice the weight of reinforcing mesh used in normal reinforced concrete pavement in Ontario; Sections E and F contain approximately $1\frac{1}{2}$ times that amount. The 4 sections were designed to measure the influence of concrete base thickness, weight of reinforcing mesh, and thickness of asphalt overlay on the spacing and width of cracks in the pavement.

The two 10-ft expansion wedges at the extremities of sections C and F were designed to accommodate the expansion of the 4 sections (5,000 ft) of continuously reinforced composite pavement.

Section G represents the normal 1959 reinforced concrete jointed pavement with 1 in. less thickness of concrete and a $3\frac{1}{4}$-in. thickness of asphalt overlay. No attempt was made to extend the contraction or expansion joints (99-ft centers) in the concrete through the asphalt.

Section H is a 1,250-ft stretch of continuously reinforced 8-in. concrete pavement with approximately twice the amount of mesh used in the normal reinforced concrete pavement. There are no joints. This section was designed to show whether a substantial quantity of reinforcing mesh (0.394-in. diameter wires at 4-in. centers) could prevent the cracks in the concrete from opening beyond a hairline width, although it was recognized that the longitudinal steel area provided (0.38 percent) was considerably less than that normally recommended (0.6 percent) for continuously reinforced concrete pavement (3). The effect of a thin asphalt overlay on the cracking propensity of the continuously reinforced concrete pavement can be made by comparing section C with section H.

Section I is normal reinforced concrete pavement (9 in. thick) based on the standard 1959 design of the Ministry of Transportation and Communications for that type of pavement.

The 24-ft wide pavement (both concrete and asphalt) was constructed in two 12-ft wide lanes. Hook-bolt dowels ($^5/_8$ in. in diameter at $2\frac{1}{2}$-ft centers) as used in the standard reinforced concrete pavement design (section I) were also incorporated into sections A, C, D, and H across the longitudinal lane joint in the concrete.

Dimensions and weight of the 3 types of reinforcing mesh used in the pavement are given in Table 2.

Quality Control Tests

In general, the compaction of the subgrade and the granular base courses was acceptable, and tests on the granular materials in the concrete and asphalt indicated that they met the specification requirements and were batched in the specified proportions.

Bearing Capacity of Base Course and Subgrade

The relative bearing value or CBR of the granular base course and the clay subgrade was determined on laboratory-compacted specimens. One test of each material was carried out on combined samples taken from 3 sections along the inside shoulder of the experimental pavement. The field sampling was carried out in November 1972. CBR values (0.1-in. penetration) were as follows:

Material	Modified AASHO Test	Standard AASHO Test
Granular base course		
Class A	68	20
Class B	130	26
Clay subgrade	3.2	2.5

The field sampling of the granular base course materials did not indicate an obvious demarcation between class A and class B. That contamination of the class B material probably accounts for its higher CBR value. Under most conditions class A material will have a higher bearing capacity than class B material.

It is apparent that the granular base course and the clay subgrade materials have an adequate bearing capacity for the types of pavement evaluated in this experiment.

LOCATION AND TRAFFIC

The experimental pavement is located on Highway 401 approximately 25 miles west of Toronto and occupies slightly more than 2 miles of the westbound lanes of the heavily traveled, dual highway. The east end of the pavement abuts the Oakville Creek structure. The subgrade throughout is uniform clay till. The grade is almost flat; the cut and fill sections are very shallow. No interchange interrupts the test area, and uniform traffic volumes exist throughout.

The annual average daily traffic volumes in the 2 westbound lanes as determined by a permanent counting station that has operated east of the project for the past 10 years is as follows:

YEAR	AADT	YEAR	AADT
1962	11,600	1968	20,000
1963	12,400	1969	22,600
1964	13,300	1970	24,200
1965	15,000	1971	27,500
1966	15,900	1972	29,000
1967	17,400		

The pavement is located east of a group of large gravel pits and limestone quarries that supply aggregates to the Toronto market. The experimental sections are located on the return route of the trucks that carry the aggregate; thus, although the truck traffic volume is high, a considerable portion consists of empty trucks. Manual counts of commercial vehicles taken in 1961, 1966, and 1972 (expressed as a percentage of total vehicles on the 2 westbound lanes) are as follows:

Year	Daytime Count Duration	Percent
1961	8	19
1966	8	24
1972	3 to 9	22

The percentages for 1966 and 1972 included 8 and 7 percent empty gravel trucks respectively.

The percentages of vehicles in the driving and passing lanes in 1972 are as follows:

Vehicle	Driving	Passing	Both
Passenger	63	93	78
Commercial	37	7	22
Empty gravel trucks	11	3	7

During a 24-hour period in July 1972, axle weights were determined by a Viatec axle weight analyzer (2). From those data, a truck factor was calculated by dividing the total number of equivalent 18-kip single axle loads by the total number of vehicles counted. The truck factor so determined was 0.61. In contrast the truck factor was 1.63 on the 2 eastbound lanes carrying loaded gravel trucks and adjacent to the experimental pavement.

COST OF PAVEMENT SECTIONS

The relative cost of constructing the various pavement sections based on 1972 prices in the Toronto area is given in Table 3. The cost index includes all construction above the top of the granular base course.

The difference between the lowest initial cost sections (A and D) and the control section (I) represents approximately $7,300 per mile of 24-ft wide pavement.

Although the cost data given in Table 3 are useful in comparing the overall economy of the 9 sections, a comparison must also be made with the cost of more modern pavement designs. A standard rigid pavement consisting of a 9-in. unreinforced concrete slab on a 6-in. cement-treated granular base and having closely spaced, skewed, transverse joints is initially considerably more expensive than the pavement in section I; the cost index is approximately 108 percent. In contrast, a deep-strength, flexible pavement consisting of 12-in. asphalt on a 9-in. granular base has a much lower initial cost than section I pavement; the cost index is in the order of 75 percent. Although the life serviceability of the unreinforced concrete pavement on the cement-treated granular base and the deep-strength flexible pavement cannot yet be established because long-term performance from either of those designs is not available, performance to date indicates that those 2 modern designs will perform adequately in a structural sense on major heavily trafficked highways.

PAVEMENT PERFORMANCE JULY 1972

In addition to the regular surveys during the first 3 years already reported (1) and an inspection during the winter of 1964-65, a comprehensive condition survey was made in the summer and fall of 1972. That included measurements of number and width of cracks, road roughness, and skid resistance. Samples were cut from the pavement

Table 1. Materials used in pavement structure.

Part of Pavement	Material	Amount
Concrete	Cement, lb/yd^3	569
	Max coarse aggregate size, in.	1½
	Fine aggregate, percent by weight of total aggregate	37
	Specified min compressive strength	
	psi	3,500
	Days	28
	Specified flexual strength	
	psi	550
	Days	10
HL1 asphalt surface	Max coarse aggregate size, in.	½
	Fine aggregate, percent by weight of total aggregate	55
	Asphalt cement	
	Penetration grade	85 to 100
	Percent by weight of total mix	5.7
HL6 asphalt binder	Max coarse aggregate size, in.	¾
	Fine aggregate, percent by weight of total aggregate	42
	Asphalt cement	
	Penetration grade	85 to 100
	Percent by weight of total mix	5.3
Macadam asphalt binder	Coarse aggregate	
	Max size, in.	1½
	Retained on No. 4 sieve, percent	90
	Asphalt cement	
	Penetration grade	85 to 100
	Percent by weight of total mix	3.5

Table 2. Reinforcing mesh used in concrete base and pavement.

Weight of Mesh ($lb/100\ ft^2$)	Longitudinal Wires			Transverse Wires			Pavement Section
	Gauge	Centers (in.)	Diameter (in.)	Gauge	Centers (in.)	Diameter (in.)	
149	4-0	4	0.394	No. 3	12	0.244	C, D, H
108	4-0	6	0.394	No. 3	12	0.244	E, F
75	2-0	6	0.331	No. 4	12	0.225	G, I

Table 3. Estimated relative initial cost of pavement sections.

Pavement Section	Construction	Cost Index[a] (percent)
A	3¼-in. asphalt over 8-in. unreinforced concrete base	94
B	4-in. macadam and asphalt over 8-in. unreinforced concrete base	97
C	1½-in. asphalt over 8-in. reinforced concrete base	102
D	1½-in. asphalt over 7-in. reinforced concrete base	94
E	3-in. asphalt over 8-in. reinforced concrete base	105
F	3-in. asphalt over 7-in. reinforced concrete base	97
G	3¼-in. asphalt over 8-in. jointed, reinforced concrete base	105
H	8-in. continuously reinforced concrete pavement	99
I	9-in. jointed reinforced concrete pavement	100

[a]Based on 1972 prices in Toronto area for large contract.

for use in examining the condition of the asphalt and determining the performance of the concrete base. The granular material forming the inside shoulder of the pavement was removed at several locations, and the condition of cracks in the composite pavement was examined. Samples of the granular materials and the clay subgrade were removed and tested to determine the CBR.

The number of transverse cracks expressed as a function of pavement age and the distribution of cracks in each section after 13 years are shown in Figures 2 and 3 respectively. Measurements of road ridability are given in Table 4, and skid resistance is given in Table 5.

After 13 years of heavy traffic, including 4 winters when the proportion of cars fitted with studded tires ranged from 8 to 30 percent, the various sections of the experimental pavement are, in general, in a satisfactory condition. An exception is the skid resistance of the 2 exposed concrete sections. A general rating of the sections is given in Table 6.

The maintenance work carried out on the pavements consisted of sealing or patching cracks in the asphalt overlay and the concrete pavement and patching about 1,600 yd^2 of asphalt mainly in 3 areas. Patching in section A was necessary because of settlement of the fill behind the abutment of the structure at the east end of the section, patching in section H was necessary when a number of wide-stepped cracks resulted in a rough ride, and patching in section C was due to undetermined reasons. The three 10-ft expansion wedges adjacent to sections B and G have not performed well and have been patched several times during the past 13 years.

The main defects evident in the pavement at the present time can be summarized as follows:

1. Wide transverse cracking accompanied in some cases by faulting,
2. Longitudinal joint cracking,
3. Breakdown of the macadam asphalt mix in section B, and
4. Low skid resistance of exposed concrete sections.

Transverse Cracking

Each section of the pavement is affected by transverse cracking; little change occurred in either number or width of cracks after the first 2 winters. The exceptions are section B, where the number of cracks has more than doubled during the past 8 years because of a breakdown of the macadam asphalt binder course, and section I, where most cracks became visible in the reinforced concrete pavement after 3 to 5 years. Because of edge raveling, with subsequent loss of paving material, the cracks in the asphalt surface course are generally considerably wider than the cracks in the concrete base. Although the cracks vary in width and spacing within and between each section of experimental pavement, each of the 7 sections of composite pavement contains a number of wide cracks.

Cracks are least frequent but widest in sections having an unreinforced base or a reinforced base with formed joints (A, B, and G). Those are shown in Figures 4 and 5. Cracks are most frequent and generally the narrowest on sections with concrete base or concrete pavement having the greatest weight of reinforcing mesh (C, D, and H). Several cracks in the concrete slab of section H are very wide and faulted (Fig. 6). The longitudinal steel area provided in that section (0.38 percent of concrete) is considerably less than that normally recommended (0.60 percent) for continuously reinforced concrete pavement (3).

A comparison between sections C and H indicates that the effect of a 1½-in. asphalt overlay on the cracking in a reinforced concrete base slab is to reduce the number of transverse cracks by about a third. That may be due to reduced flexural stresses in the concrete base of the thicker pavement, lower stresses in the concrete base from slab curling and slab shortening due to temperature reductions, and lower drying shrinkage stresses in the concrete base.

A slightly less valid comparison can be made between sections G and I. The effect of a 3¼-in. thick asphalt overlay on a relatively lightly reinforced concrete section (G) that is 1 in. less thick than the exposed concrete section (I) was to eliminate almost all intermediate cracking between the 99-ft joint spacing.

Figure 2. Transverse cracks as a function of pavement age.

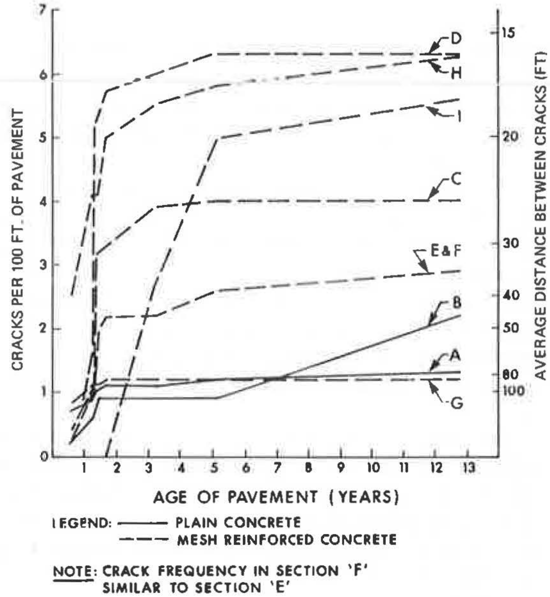


Figure 3. Distribution of crack widths after 13 years.

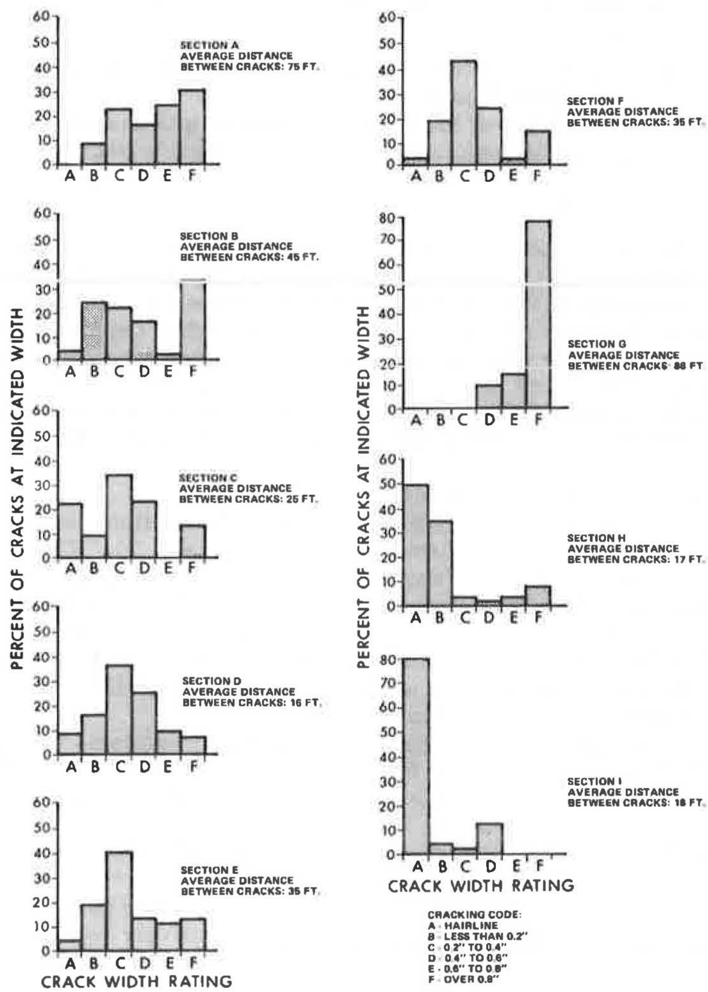


Table 4. Road ridability in July 1972.

Pavement Section	R[a] (in./mile)	Surface	Subjective Classification of Ridability[b]
A	69	Asphalt	Good
B	92	Asphalt	Average
C	76	Asphalt	Good
D	95	Asphalt	Average
E	77	Asphalt	Good
F	99	Asphalt	Average
G	71	Asphalt	Good
H	97	Concrete	Fair
I	70	Concrete	Good

[a]Road ridability measured in driving lane by BPR roughometer trailer.
[b]Based on road user's evaluation of Ontario pavement (5, 6).

Table 5. Skid resistance in July 1972.

Pavement Section	Skid Number	Surface
A	33.0	Asphalt
B	33.0	Asphalt
C	33.0	Asphalt
D	31.5	Asphalt
E	35.8	Asphalt
F	40.0	Asphalt
G	35.8	Asphalt
H	20.8	Concrete
I	20.0	Concrete

Note: Skid resistance measured in driving lane by ASTM brake force trailer at 60 mph.

Table 6. General rating of pavement sections.

Pavement Section	Riding Quality	Skid Resistance	Initial Cost	Maintenance Cost
A	Good	Good	Lowest	Low
B[a]	Average	Good	Below average	High
C	Good	Good	Above average	Average
D	Average	Good	Lowest	High
E	Good	Good	Highest	Average
F	Average	Good	Below average	Average
G	Good	Good	Highest	Low
H[a]	Fair	Poor	Average	High
I	Good	Poor	Average	Low

[a]Will need resurfacing in near future.

Figure 4. Wide transverse crack in asphalt pavement on unreinforced concrete base (section A).

Figure 5. Wide transverse crack in asphalt pavement over contraction joint in reinforced concrete base (section G).

Figure 6. Wide stepped transverse crack, about 3½ in. wide, in continuously reinforced concrete pavement (section H).

In sections C, D, E, and F, representing either $1\frac{1}{2}$ or 3 in. of asphalt over 7 or 8 in. of reinforced concrete base, the effect of increasing concrete thickness was to reduce the cracking by about a third in slabs containing the heavier weight of mesh; however, the effect of thickness on the number of transverse cracks in sections containing 108 lb of mesh per 100 ft^2 was negligible. In slabs of constant thickness, an increase in the amount of steel mesh from 108 to 140 lb per 100 ft^2 resulted in an increase in the number of cracks by about 80 percent and a slightly decreased average crack width.

The thickness of asphalt overlay on sections A to G varied from $1\frac{1}{2}$ to 4 in. In no case does it appear that the thickness of asphalt has had any influence on the amount of cracking reflected through from the concrete base. Wherever a crack is observed in the concrete base, there is directly above it a reflected crack in the asphalt overlay.

Concrete pavement elsewhere in Ontario similar to section I in design has shown only very limited reflected transverse cracking when resurfaced with 5 in. of asphalt pavement. That pavement overlay is approximately 4 years old.

Transverse cracking in bituminous pavements placed directly on a granular base course has been identified as a serious problem in the northern parts of Ontario, where the freezing index is high (4). Such cracks permit the ingress of water and salt solutions, which lead to stripping of the bituminous mix, pumping of fine granular base material, and local weakening of the pavement structure. The transverse cracking occurring on the sections of composite pavement is seemingly less of a problem; if properly maintained, it will not seriously affect the riding qualities of the pavement, unless vertical faulting of the crack occurs. Many of the wider cracks in the pavement are faulted. In sections D and H, where about half of the transverse cracking exhibits faulting, it is undoubtedly a major cause of the relatively high R values as determined by the roughometer trailer. Most "steps" are in the range of $\frac{1}{8}$ to $\frac{1}{4}$ in.; the highest one measured was $\frac{3}{4}$ in. at a wide crack in section D. Removal of the shoulder material adjacent to a wide faulted crack in section D indicated severe spalling of the bottom half of the 7-in. thick concrete slab (Fig. 7). The crack width in the concrete slab was $1\frac{3}{4}$ in. The 4-0 gauge longitudinal wires of the reinforcing mesh were "necked" and broken at the crack. Less severe spalling was also observed at narrower cracks in the concrete base (Fig. 8).

The 4 mesh reinforced composite sections (C to F) exhibited faulting to a degree, and neither the weights of mesh nor the thickness of asphalt overlay appears to have much influence on the severity of faulting. There is, in general, less faulting in sections with an 8-in. concrete base (C and E) than in sections with a 7-in. concrete base (D and F). The composite sections (A and B) with an unreinforced concrete base show little or no stepping despite the fact that the cracks in the unreinforced base are considerably wider than the cracks in the reinforced base pavement.

Longitudinal Joint Cracking

In the sections with an asphalt overlay, wide cracking of the centerline joint is a general feature of the pavement. In some cases the original center crack in the bituminous pavement has been followed by parallel secondary cracks leading to loss of pavement material in widths as great as 6 in.

The hook-bolt dowels in 3 of the 7 bituminous-covered sections had little effect on the severity of the cracking. Both doweled centerline joints with $1\frac{1}{2}$-in. thick asphalt (Fig. 9) and undoweled centerline joints with 8-in. thick asphalt (Fig. 10) are associated with wide cracks and loss of bituminous pavement.

The longitudinal, open crack is in extreme cases a traffic hazard unless patched; but, potentially, the most important influence of the cracks on pavement performance is their ability to allow water into the bituminous binder course and thus to accelerate stripping of the mix.

The joint, apart from one short superelevated section, is at the center of the 2-lane pavement, which has a cross fall to both shoulders.

It is apparent that the longitudinal joint in the concrete base is open (Fig. 9) and that the hook-bolt dowels ($\frac{5}{8}$-in. diameter at $2\frac{1}{2}$-ft centers) have not held the two 12-ft wide slabs together. A core taken through the pavement at the location of a hook-bolt

Figure 7. Severe spalling in bottom half of concrete base slab at wide crack (section D).

Figure 8. Less severe spalling near base of concrete slab at narrow crack (section D).

dowel indicated that the assembly had failed and that 1 hook bolt had pulled out from the threaded coupling because of partial fracture and distortion of the coupling. A $^3/_8$-in. wide gap exists between the 2 vertical faces of the concrete. Laboratory tests on a sample of hook-bolt dowel assemblies similar to those used in the pavement indicated that under tension the assembly failed at a load of 8.06 tons. The fracture occurred in the coupling connecting the 2 hook-bolt dowels. The $^5/_8$-in. diameter hook bolts failed in tension at a load of 9.15 tons.

The purpose of hook-bolt dowels in single-lane paving is to hold the longitudinal joint tightly closed. It appears in this pavement that the dowels are not adequate for that purpose and should be either strengthened or used at closer centers in future work.

Macadam Bituminous Mix in Section B

In section B a considerable amount of random cracking (Fig. 11) on the pavement surface has developed during the past few years. During that time, and contrary to the trend of the other sections, the number of transverse cracks has more than doubled. Some rutting of the bituminous surface has occurred in the wheelpaths, and there appears to be some loss of shape on the roadway caused by deformation of the bituminous layers.

Samples of the bituminous mix cut out of the pavement revealed that the random cracks were confined to the HL1 surface course and did not occur in the concrete base. The macadam bituminous mix is badly stripped (Fig. 12). The stripping has advanced to the point that, near cracks, the mix has lost most of its structural strength and can be readily broken with the fingers. Stripping was first observed during a sampling program when the pavement was 6 years old. At that time, the stripping had not adversely affected pavement performance, and the cracks in the asphalt were almost all reflection cracks from the concrete base. The macadam mix sampled at several locations was quite wet and contained a lot of dirt and fines in the matrix. Because very little water will normally penetrate through the HL1 surface course, it is probable that water in the uncracked sections of the pavement has penetrated from adjacent cracks, particularly the centerline crack. One sample of bituminous pavement taken the day following a heavy rainfall revealed the macadam mix to be saturated, and water was pooled on the surface of the concrete base.

The widespread random cracking and some of the transverse cracking in section B are due to stripping and breakdown of the 3-in. thick macadam binder course. That type of macadam binder course has been used as part of an asphalt overlay on several miles of concrete pavement on Highway 401 near the experimental pavement and after 4 years of service appears to be in excellent condition. Nevertheless, the condition of the macadam in section B indicates that ways must be found to keep the material in a relatively dry condition if successful long-term performance is to be attained.

Skid Resistance

The measurements of skid resistance (Table 5) indicate that sections A to G with an HL1 bituminous mix surface course have acceptable skid characteristics after 13 years of service. The average skid number of 34.6 provides good friction for high-speed traffic on the relatively straight alignment occurring on the pavement. The bituminous surface course has retained its coarse texture, and the traprock particles protrude above the remainder of the mix.

The original burlap-drag surface texture on the concrete sections H and I has been worn off by the action of studded tires, and the limestone coarse aggregate is exposed in the wheelpaths. The limestone particles have subsequently polished and caused a significant decline in skid resistance of the exposed concrete sections.

Granular Base Course

An examination of the concrete base in November 1972 involved the removal of shoulder material down to the underside of the concrete slab along 200 ft of roadway. The granular material near the edge of the pavement was saturated, and water rapidly pooled at the surface of the excavation (Fig. 13). That surface water is also visible in Figure 8. Rainfall during the 7 days prior to this work was 1.1 in.

It is apparent that, in spite of a 12-in. thickness of granular material under both the concrete slab and the crushed-stone shoulder, water from the roadway surface is not drained from the structure of the pavement but collects readily in the base and subbase. Pavement damage to composite or concrete pavements from frost heave is not common in southern Ontario; nevertheless, the high moisture content of the subgrade with the possibility of ice-lense formation during the wintertime is potentially a cause of pavement faulting, particularly at wide transverse cracks.

Summary

The general criteria usually used to rate the present performance of a pavement are riding quality and skid resistance. The economical aspects are considered as initial costs plus the cost of maintenance. Ratings were applied to each of the 9 sections to show the overall economy and performance of each section (Table 6). The cost of maintenance is based on the amount of cracking and other defects visible in the pavement at the present time.

It is considered that section B, due to pronounced random cracking and loss of shape of the bituminous overlay, and section H, due to the stepping of numerous cracks, will need to be resurfaced in the near future.

In a general sense, section A (3¼ in. of asphalt over an 8-in. unreinforced concrete base) is considered to be the most satisfactory of the experimental sections. The present performance of that section, i.e., riding quality and skid resistance, is good, and its initial and estimated maintenance costs rank lowest of all the sections.

Sections C, E, and G also have satisfactory performance, but initial cost and in 2 cases the estimated maintenance costs are greater than those for section A.

The standard reinforced concrete pavement section (I), the control pavement on this experiment, exhibits good riding quality and low estimated maintenance costs. Only the skid resistance makes the section unsatisfactory.

A study confined to a comparison between the experimental sections and section I is of limited usefulness because the 1959 design of reinforced concrete pavement (I) is not now considered a suitable pavement for a heavily trafficked, controlled-access highway. A more useful comparison is with other freeway pavements in southern Ontario, in particular the other 508 miles of Highway 401. Although precise comparisons are plainly impossible because of variable traffic conditions and age of the pavement (most of Highway 401 is between 7 and 15 years old), it becomes apparent that the best composite pavement (section A), which has carried heavy traffic for 13 years and is still a good pavement, performs better than most other rigid or flexible pavements.

CONCLUSIONS

After 13 years of service under heavy traffic on a dual highway, the major problems evident on the experimental pavements are longitudinal and transverse cracking of the bituminous surface course, breakdown of the macadam binder course used in one section, and low skid resistance of the exposed concrete sections. The most satisfactory pavement is section A, 3¼ in. of bituminous overlay on an 8-in. unreinforced concrete base. The important findings of these experimental pavements are summarized below:

1. The thicknesses of bituminous overlay (up to 4 in.) were not adequate to bridge cracks or joints in the concrete base.
2. The asphalt overlay on a reinforced concrete base significantly reduced the number of transverse cracks in the concrete slab.
3. The effect of the reinforcing mesh in the concrete base slab was to increase the number and decrease the width of the cracks. In most sections, several very wide stepped cracks indicate failure of the mesh and loss of load transfer at the joint. Reinforcing mesh in the concrete base appears to have no beneficial effect on the overall performance of the composite pavements.

Figure 9. Loss of asphalt pavement due to cracking at centerline joint (section D).

Figure 10. Loss of asphalt pavement due to cracking at centerline joint (section F).

Figure 11. Random cracking in asphalt surface course (section B).

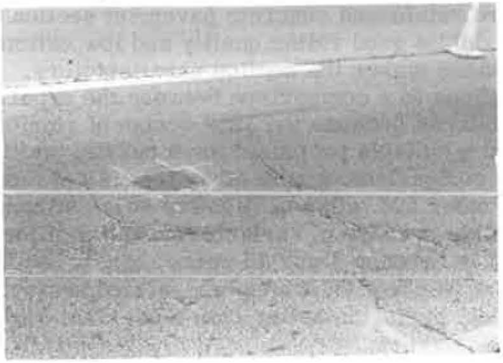


Figure 12. Breakdown of macadam mix binder course due to stripping of asphalt film from coarse aggregate particles (section B).

Figure 13. Surface water in granular shoulder at edge of pavement.

4. Unless ways can be found to reduce the stripping of the asphalt cement film from the coarse aggregate particles, the macadam bituminous mix used in section B is not a satisfactory binder course for this type of pavement.

5. The limestone coarse aggregate used in the concrete pavement sections became exposed and polished, resulting in a roadway with low skid resistance. That condition is mainly due to the extreme abrasive conditions that existed when studded tires were used in Ontario.

6. The continuously reinforced concrete pavement (0.38 percent longitudinal steel) in section H had the poorest ridability characteristics of all the sections because of stepping at many of the transverse cracks.

The superiority of the best composite section (A) over the control reinforced concrete section (I) is due to the superior skid resistance and lower initial cost of the composite section. On that basis, composite pavement offers potential advantages over conventional reinforced concrete pavement for heavily trafficked expressway pavements.

Better performance from composite pavement may be achieved if the wide transverse cracks, present in each of the experimental sections, are prevented from occurring. That may be accomplished by the use of transverse crack inducers in the concrete base at approximately 15-ft centers. To offset such extra costs, the economic advantages of composite construction over conventional concrete pavement might be maintained through the use of a lower quality concrete in the base and less rigid requirements for surface smoothness and the use of placing and finishing equipment.

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