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Performance of Composite Pavement, Overlays, and Shoulders

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Subject Areas

25 Pavement Design
26 Pavement Performance
40 Maintenance, General

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CONTENTS

FOREWORD

The papers presented in this RECORD will be of great interest to those interested in unusual pavement designs.

Ryell and Corkill present an evaluation of an experimental pavement constructed in 1959. Three basic pavement designs are described: unreinforced concrete base with an asphalt overlay, mesh reinforced concrete base with an asphalt overlay, and continuously reinforced concrete pavement. Included are data on the transverse and longitudinal cracking, road ridability, and skid resistance after 13 years of service. The authors conclude that the most satisfactory pavement section in the experiment consists of a 3¼-in. bituminous overlay on an 8-in. reinforced concrete base.

Baker reports on a composite pavement section constructed in 1963 on a major New Jersey route. The pavement is a 5-component system involving a 6-in. crushed-stone subbase, an 8-in. plain concrete base, 3 in. of densely graded crushed stone, a 5-in. layer of dry-bound macadam, and a 3½-in. surfacing of bituminous concrete. Performance data are presented for this unusual pavement and include deflections, rut depths, roughometer measurements, cross-sectional and profile measurements, and condition surveys.

Martin presents the major factors affecting the design and construction of concrete resurfacing. He points out that limiting slab deflection is more important than limiting slab stress. Maximum slab deflection for various methods of load transfer across joints or cracks is proposed. Longitudinal steel percentages are suggested for various slab temperatures.

Rafiroiu gives a brief report on 2 composite-pavement experiments in Romania. After several years of heavy traffic, no cracks were observed in the pavement surfaces.

Ramsamooj deals with the problem of predicting the rate at which cracks in an underlying layer of pavement will reflect through a bituminous overlay. The theories of both linear elastic fracture mechanics and delayed fracture in viscoelastic materials are used to formulate a method of solution.

Lokken summarizes the performance to date and examines design details developed from experimental projects using PCC shoulders. He recommends that designs include maximum safety and economy. The paper also suggests further areas of study in the design of concrete shoulder-roadway systems.

—George B. Sherman

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 3¼-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.

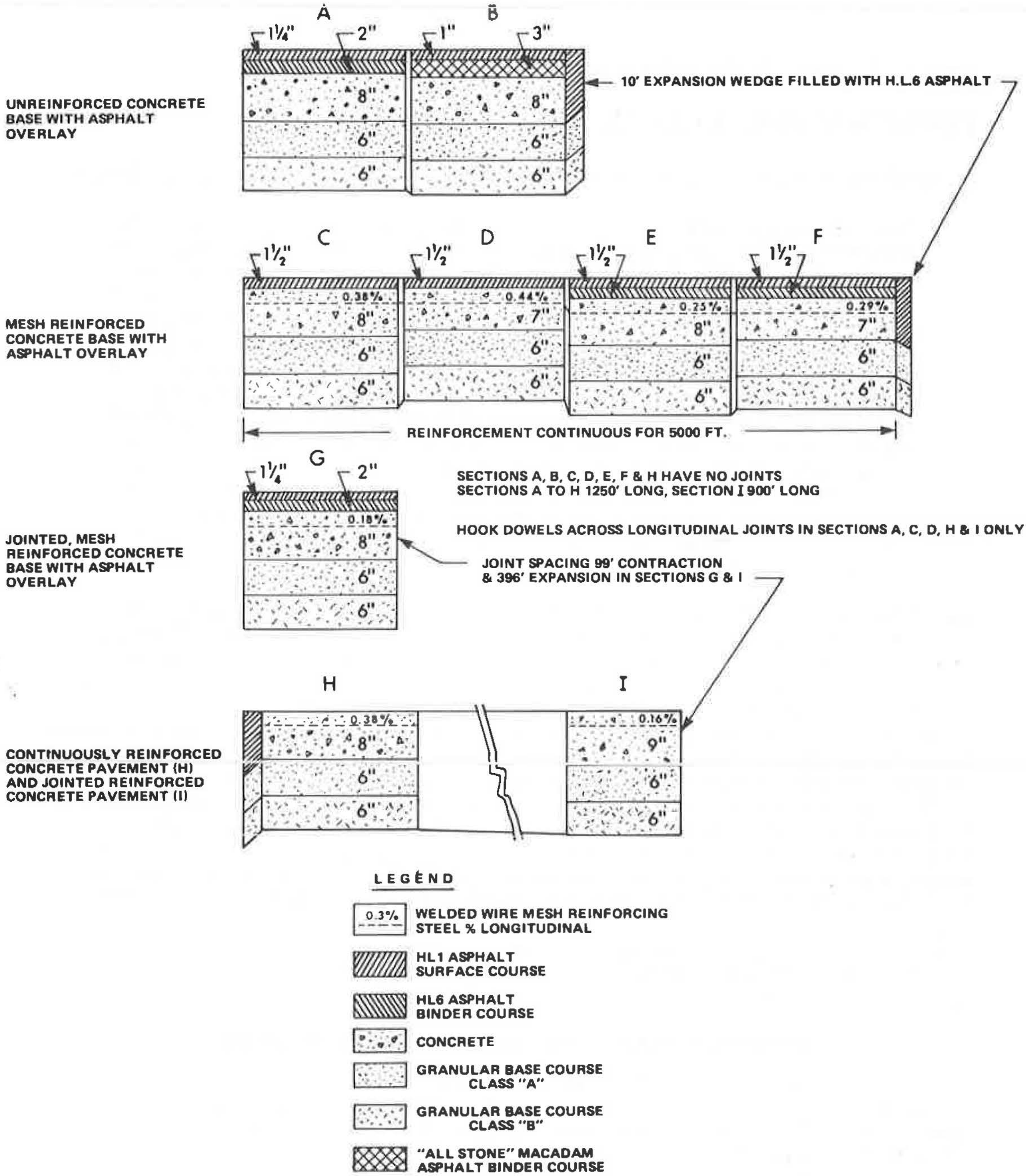
Details of the experimental pavements together with performances during the first 3 years were previously reported (1). This report describes the performances of the pavements after almost 13 years of service.

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 Gauge	Longitudinal Wires Centers (in.)	Longitudinal Wires Diameter (in.)	Transverse Wires Gauge	Transverse Wires Centers (in.)	Transverse Wires Diameter (in.)	Pavement Section
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.

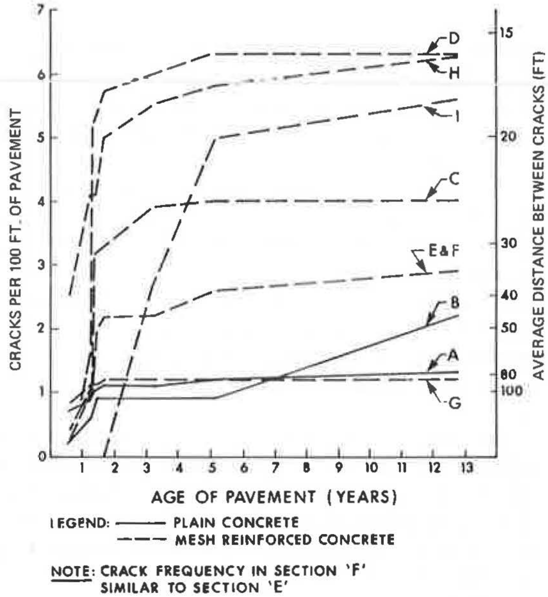


Figure 3. Distribution of crack widths after 13 years.

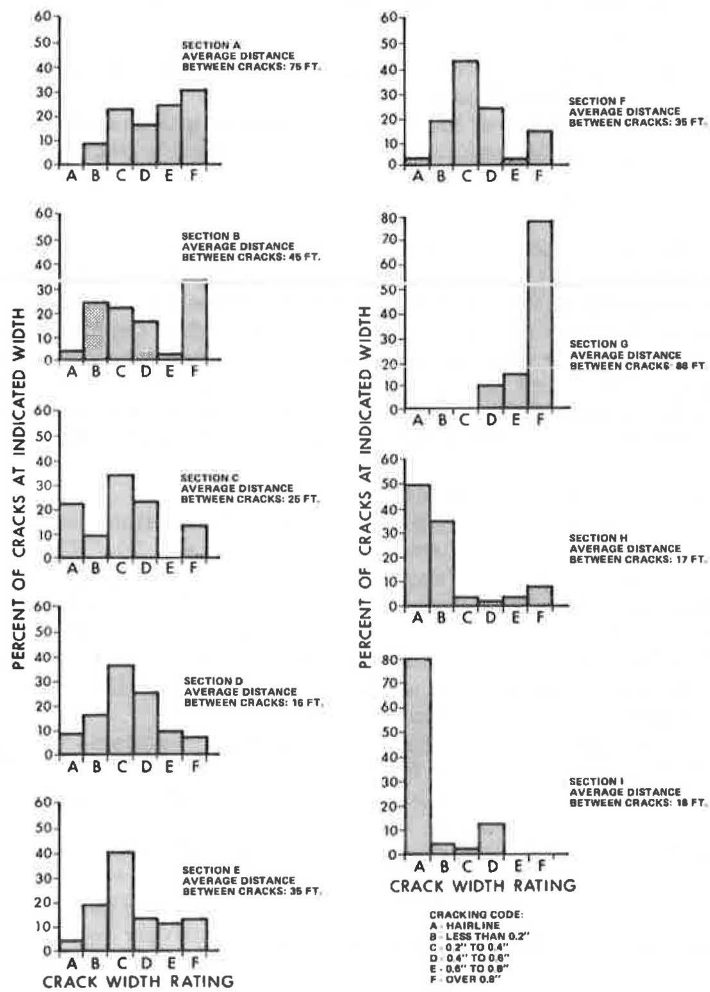


Table 4. Road ridability in July 1972.

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 1½ 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 1½ 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 ⅛ to ¼ in.; the highest one measured was ¾ 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 1¾ 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 1½-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 (⅝-in. diameter at 2½-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 ($3\frac{1}{4}$ 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, $3\frac{1}{4}$ 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).

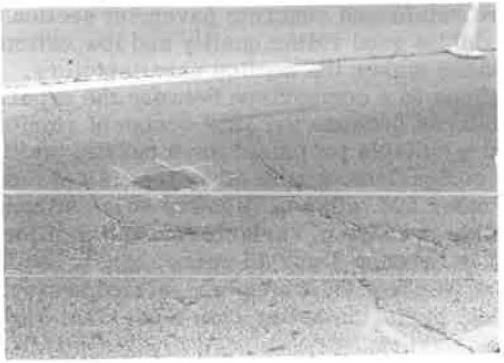


Figure 12. Breakdown of macadam mix binder course due to stripping of asphalt film from coarse aggregate particles (section B).

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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NEW JERSEY COMPOSITE PAVEMENT PROJECT

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This paper reports on the 8-year performance of a 1,500-ft section of composite pavement constructed in 1963 on NJ-3, a major route carrying approximately 88,780 vehicles daily: Of those vehicles, 14.6 percent are trucks. The pavement consisted of a 5-component system involving a 6-in. crushed stone subbase, an 8-in. plain concrete base, a 3-in. layer of densely graded crushed stone, a 5-in. layer of dry-bound macadam, and a 3½-in. surface of bituminous concrete. The pavement was designed to provide long-term serviceability and to require an absolute minimum of maintenance, despite the unstable nature of the terrain and very heavy traffic. Cost was about 15 percent higher than that of New Jersey's standard design of reinforced concrete pavement but is expected to be offset by the estimated 23 percent longer service life. Performance data from Benkelman beam deflections, rut depths, roughometer measurements, cross-sectional and profile measurements, and condition surveys show that the composite pavement has performed as designed with minimal maintenance.

•THIS REPORT analyzes the performance of an experimental composite pavement on the westbound roadway of NJ-3 at the east approach to the Hackensack River bridge during the period from July 1963 to June 1971. It also includes condition surveys of similar pavement on the other approaches and of a somewhat thinner composite pavement on the NJ-3 connection to NJ-20 northbound.

The basic data analysis for the pavement on the westbound roadway consisted of the following: traffic data, including loadings; deflection measurements; settlement determinations; concrete base performance; and bituminous concrete surface performance.

DESIGN AND CONSTRUCTION

Several factors influenced the use of a composite design rather than a conventional design in the bridge approach sections:

1. A portion of the terrain over which this highway was constructed was meadowland, and the subsoils consisted of thin varied layers of silt, clay, and sand.
2. The high daily volume of vehicles and a large percentage of heavy truck traffic required that the pavement section be of an unusually sturdy construction in order to remain trouble-free and require minimum maintenance during a relatively long period of time.

The usual procedure for embankment construction in a meadowland area is to preconsolidate the subsoils with the use of sand drains and an overburden. That procedure, unfortunately, requires a minimum settlement time of several months. Because of the high volume of vehicles and the excessive deterioration of the old bridge, which was approaching a hazardous condition, the lengthy settlement period had to be eliminated. Accordingly, no preconsolidation was attempted. Construction proceeded with the removal of the organic peat and muck and replacement with an open-graded quarry-processed material. A normal earth embankment and pavement construction were then placed above that.

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

New Jersey's standard design of reinforced concrete pavement (78-ft, 2-in. slab length having doweled expansion joints) did not appear suitable for the project because of the compressible nature of the underlying soil and the possibility of appreciable differential settlements causing serious cracking. The past performance of flexible pavements on major trucking routes in New Jersey meadowland raised doubts as to the adequacy of any conventional flexible pavement under the extreme traffic and unstable soil conditions in that area.

Past experience indicated that bituminous surfacing materials are capable of satisfactorily carrying heavy truck traffic, provided that the bituminous materials are on a highly stable foundation. Two rehabilitation projects that involved thick overlays placed on existing concrete pavement (US-22 between Somerville and Chimney Rock and US-130 in the vicinity of Deans) provided outstanding performance. That performance served as the prime basis for the selection of a composite pavement design for the Hackensack River site.

The objectives in using a composite pavement were

1. To maintain the structural integrity of surface despite an anticipated differential settlement resulting from the deep fill and compressible nature of the underlying soil;
2. To achieve the high load-carrying capacity of a rigid pavement necessitated by the large volume of heavy truck traffic; and
3. To achieve the continuity of surface of a flexible pavement and to minimize reflection cracking.

The specific design selected to achieve those objectives is shown in Figure 1. A 6-in. layer of quarry-processed stone was placed on the subgrade. An 8-in. plain portland cement concrete base (New Jersey class D mix, 1:2.25:4.5) was placed on the subbase and was overlaid with a 3-in. course of densely graded stone base and a 5-in. lift of dry-bound macadam base ($2\frac{1}{2}$-in. stone choked with stone screenings). The composite pavement section was completed with a $3\frac{1}{3}$-in. layer of bituminous concrete (FA-BC-2) consisting of a 2-in. bottom or binder course and a $1\frac{1}{2}$-in. surface course.

The pavement was constructed in July 1963. It is approximately 1,500 ft in length and is divided into two 12-ft lanes and two 13-ft lanes. A concrete curb abuts the edge of the inside lane. A bituminous shoulder abuts portions of the outside lane.

The NJ-20 ramp was of similar construction, except that a 6-in. plain concrete base was used instead of an 8-in. base. The plain concrete base at both locations was constructed with undoweled contraction joints at 15-ft intervals and no expansion joints. Longitudinal joints between lanes were of the plain butt type without tie bars.

The 3-in. base of densely graded stone was placed between the concrete base and the 5-in. macadam base in order to act as a buffer to prevent the reflecting of cracks from the concrete base into the bituminous surface pavement.

COST

The construction bid price of the composite pavement was \$12.39/yd^2, which was much higher than originally anticipated. The cost difference was attributed to the contractor's inexperience with that type of pavement. The relatively short length of the section was also a factor. In subsequent contracts for the other bridge approaches, the bid price was reduced to \$9.85/yd^2. As a comparison, the bid price of a high type of bituminous concrete pavement and a reinforced portland cement concrete pavement in New Jersey, at that time, was approximately \$6 and \$9/yd^2 respectively.

The present construction bid price of the composite pavement (8-in. concrete base), as determined by current averaged weighted quantity prices for New Jersey, is approximately \$17.30/yd^2. In comparison, the same source indicates that a high type of bituminous concrete pavement and a reinforced portland cement concrete pavement cost approximately \$8 or \$15/yd^2 respectively.

The current construction bid price of a 6-in. plain concrete base is approximately 25 percent less than that for the 8-in. plain concrete base. The overall construction bid price of a composite pavement using the 6-in. plain concrete base design is approximately 15 percent less than that of one using an 8-in. concrete base and is essentially the same as the cost for reinforced portland cement concrete pavement.

TRAFFIC DENSITY

The 1970 AADT at the approaches to the NJ-3 Hackensack River bridge in both directions was 88,780 vehicles. In the westbound roadway, trucks accounted for 14.6 percent of the traffic and were distributed per lane as follows (lane 4 is inside left):

Lane	Percent
1	19
2	43
3	32
4	6

From July 2, 1963, to May 27, 1964, the present westbound roadway was used for both eastbound and westbound traffic during the construction of the present eastbound approaches to the Hackensack River bridge and rehabilitation of the southerly bridge. At that time the present westbound roadway was divided into 5 lanes, each 10 ft wide, and was subjected to 3,505,056 equivalent 18-kip axle repetitions.

After May 28, 1964, the roadway was used exclusively for 4 lanes of westbound traffic. The accumulated 18-kip equivalent axle repetitions up to and including December 31, 1970, were as follows:

Lane	Repetitions
1	2,785,704
2	6,427,714
3	4,723,743
4	904,258

ESTIMATION OF PAVEMENT LIFE

Estimates of the remaining service life of existing pavement are based on the findings of the AASHO Road Test (1). According to the Road Test findings, pavement performance is directly related to pavement structure and axle loads. The AASHO Road Test developed equations to estimate pavement performance in terms of the number of axle load applications. Those equations evolved into a simplified procedure for estimating the remaining equivalent 18-kip axle load applications for pavement service life.

The flexible pavement service life nomograph shown in Figure 2 (1) and developed from the AASHO Road Test equations was used to estimate the accumulated number of equivalent 18-kip single-axle load applications from the time that the pavement is placed in service to the time that it requires resurfacing or reconstruction (ELA_p). The calculated structural number $\overline{SN}$ of the pavement was based on the AASHO design equation ($SN = a_1d_1 + a_2d_2 + a_3d_3$), where a is the material strength coefficient and d is the layer thickness. In accordance with those equations, the structural number of the composite pavement section was estimated and is given in Table 1.

The ELA_p was estimated from the nomograph (Fig. 2) by the known soil support value ss of 6.75 for the NJ-3 location and the derived structural number of 7.62 for this pavement. However, the nomograph does not go beyond a structural number of 6.0. With that structural number, a conservative estimated service life of 65 million 18-kip applications was obtained. The composite pavement has already experienced 14.8 million 18-kip single-axle load applications. Therefore, the estimated remaining service-life was 50.2 million 18-kip axle applications.

The remaining service life after December 31, 1970, was conservatively estimated to be 30 years. The total service life, from the time of construction until overlay or replacement of the original bituminous surfacing is required, was estimated to be 38 years. Of course, this theoretical concept does not take into account deterioration of the bituminous surface course through age hardening.

ROUGHNESS SURVEY

The fourth consecutive roughometer survey, in which the New Jersey Department of Transportation roughometer was used, was completed in March 1971. For readings prior to January 1968, the BPR roughometer was used.

Table 2 gives the roughometer measurements made to date. There was little variation in the measured riding quality of the pavement in spite of the fact that data were collected with 3 separate roughometer units. There are also no discernible differences due to seasonal changes in either the pavement surface or the equipment.

A commonly applied criterion for assessing roughness data is that developed by the Federal Highway Administration from a 580-mile survey of new rural Interstate mileage in 17 states. According to that criterion, lane 1 and lane 4 rated poor in riding quality. Lanes 2 and 3 rated fair. That rating is, of course, subjective. Because of the process of extrapolating data from the actual 0.3-mile length to the indicated inches of roughness per mile, the actual roughness of the measured section tends to be exaggerated.

DEFLECTIONS

Pavement deflections were measured with a Benkelman beam in the right wheelpath of lane 1. Nine Benkelman beam surveys were conducted since the inception of the project. A 7,500-lb wheel load was used in the first 2 surveys, and a 9,000-lb wheel load in all the others. That loading corresponds to the 18-kip axle loading used in flexible pavement design. Readings were taken at the contraction joints (aggregate interlock load transfer), construction joints (no load transfer), and midpoints of the concrete base slabs.

A summary of the deflections, recoveries, and residuals to November 1970 is given in Table 3. The deflections measured to date are well within acceptable limits and do not appear to indicate a substantial increase from the original 9,000-lb load deflection and recovery readings. The upper limit (3) of acceptable values is approximately 0.020 in. for bituminous pavement.

Negative residuals recorded in the 1965 and 1966 readings were probably caused by preloading of the test slab because of the close proximity of the weighted vehicle. That indicates that the load transfer by aggregate interlocking action was functioning properly at the time. The measurements in cool weather (Oct. 1965) had lower negative values as would be expected. The absence of negative values in subsequent surveys may be due to that fact that measurements were made in cool weather or that the effective load transfer across the joints lessened.

In previous studies, Benkelman beam readings fluctuated somewhat with higher ambient temperatures and exposure to direct sunlight. Because measurements on this section were so small, the effect of those fluctuations may introduce the apparent negative residuals observed.

SETTLEMENT

Periodic close interval profile and cross-sectional readings were taken on the pavement surface in the outside and inside lanes by means of a level and rod; the readings were taken to the nearest estimated 0.001 ft. The profile readings, taken at 2-ft intervals in the outer wheelpath of the outside lane and the inner wheelpath of the inside lane, extend from station 0+00 (the junction of the pavement with the east abutment of the Hackensack River bridge) eastward to station 3+00. That was the approximate area where subsoils consisted of unstable materials.

Cross-sectional readings were taken at 1-ft intervals at each even station for the entire length of the project. The original cross sections extended across the entire roadway. However, all subsequent surveys were limited to the inside and outside lanes in an effort to minimize interference with traffic and provide for the safety of personnel.

Figure 3 shows the average profile settlement for both the inside and outside lanes. Figure 4 shows a typical cross-sectional plot for stations 1+00 and 3+00. The average profile settlement of both the outside and inside lanes has esstentially stopped; most

Figure 1. Experimental composite pavement section.

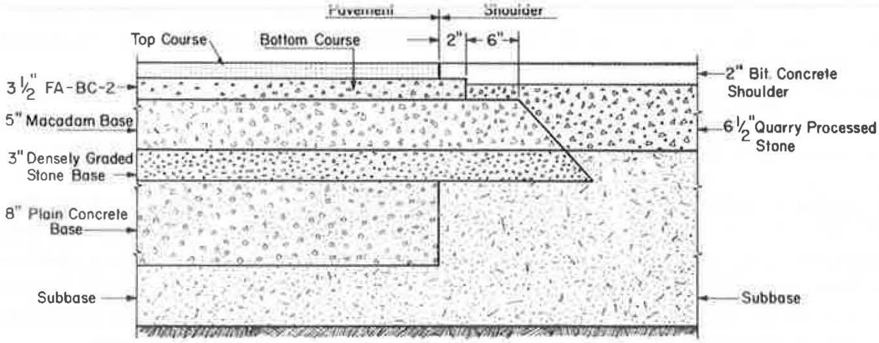

Figure 2. Flexible pavement service life nomograph (psi = 2.5).

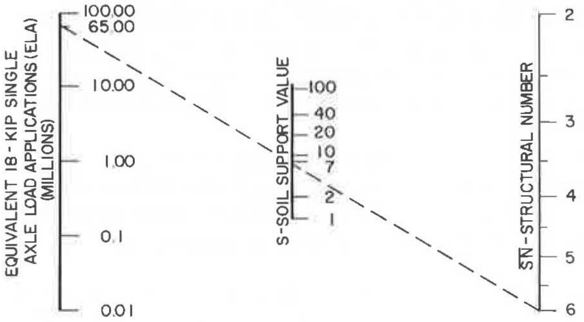

Table 1. Structural number of pavement section.

Thickness (in.)	Material	Coefficient	$\overline{SN}$
3½	Bituminous concrete	0.44	1.54
5	Macadam base	0.20	1.00
3	Densely graded stone	0.14	0.42
8	Plain concrete	0.50	4.00
6	Subbase	0.11	0.66
Total			7.62

Table 2. Road roughness survey results.

	Roughness Index (in./mile)			
Date	Lane 1	Lane 2	Lane 3	Lane 4
Nov. 1964	103	108	102	110
May 1965	110	107	105	110
Nov. 1965	110	103	103	107
Nov. 1966	106	92	101	107
Jan. 1968	116	103	110	121
Dec. 1968	100	91	96	117
Feb. 1970	115	95	102	124
March 1971	115	91	101	128

settlement occurred the first year after construction. The measured initial settlement was originally anticipated on the project. The typical cross-sectional plots show the maximum settlement in the wheelpaths. Settlement appears more prominent at station 1+00 in the proximity of the Hackensack River bridge, where as much as 0.10 ft of settlement occurred. A lesser amount of settlement occurred at station 3+00, 200 ft farther from the bridge.

RUT DEPTHS

Rut depths were taken at 50-ft intervals in both wheelpaths in lanes 4 and 1 for the entire length of the project. Rut depths were recorded as the measured distance from the bottom of an 8-ft straightedge placed across 1 wheelpath to the bottom of the rut. As with the profiles and cross sections, the rut-depth measurements were limited to the inside and outside lanes in the interest of traffic and safety. Five rut-depth surveys have been conducted to date. The average measurements are given in Table 4.

Little increase in rutting in the left wheelpath of lane 1 was noted since the original survey. However, rut-depth measurements in the right wheelpath show an increase of 0.15 in., most of which occurred between the December 1969 and November 1970 surveys. That rutting is mostly in the area of localized cracking near Grace Street and will be discussed later.

Essentially no increase in rutting in lane 4 was noted since the original survey. The large increase in rutting measured during the December 1968 survey was due to the improper selection of the wheelpath area. The relatively large rut-depth measurements in the left wheelpath are attributed to inadequate initial compaction close to the curb at the time of construction.

By the way of comparison, a survey on a standard section of flexible pavement on I-80 (constructed with a 2-in. bituminous top, 4-in. bituminous-stabilized base, 6-in. dry-bound macadam, 6-in. quarry-processed stone, and 12-in. subbase) having a comparable age but lighter traffic volumes shows average rut depths of 0.20 to 0.40 in. for the left wheelpath and 0.30 to 0.45 in. for the right wheelpath of the outer lane. In spite of the fact that NJ-3 received a greater volume of traffic, the rutting is not greater than the standard section of I-80.

CONDITION SURVEY

A detailed condition survey of the pavement surface was made in March 1971. A few fine inconsequential cracks were observed close to the catch basins and along the pavement edge but did not appear to have progressed since observed in the 1970 survey. No reflection cracks were found over the joints in the underlying concrete base.

A large crack, ¼ in. wide by 82 ft long, at the Grace Street exit is located over and parallel to a 24-in. water main. It is possible that the longitudinal cracking may be caused by differential settlement in the area of the water main.

A condition survey of the visual condition of the pavement surfaces of sections 1E, 1F, and 4G was conducted in July 1971. Those 3 approaches were partially constructed with the identical composite pavement design. However, some areas were constructed on existing concrete pavement with various thicknesses of bituminous materials.

Where the pavement was constructed with the composite pavement design, several fine reflection cracks were noted. However, at least 90 percent of the underlying joints on main-line pavement of sections 1F and 4G and section 1E do not show any reflection cracking. Insignificant patches and indentation or scarring were noted on all 3 approaches. No evidence of raveling or shoving was noted.

The NJ-20 ramp from NJ-3, part of section 1E, is constructed of a similar pavement design with the exception of a 6-in. concrete base rather than the 8-in. base as on the main-line pavement. This pavement was not constructed on a subgrade similar to that of the main-line pavement. In the 1971 condition survey of that ramp, approximately 15 percent of the underlying joints showed reflection cracks approximately ⅛ to ¼ in. wide. Those cracks are mainly in the area where NJ-3 eastbound connects with NJ-20 northbound. The distress may be attributed to a localized depression.

Table 3. Benkelman beam survey results.

Date	Avg Value at Joints (0.001 in.)			Avg Value at Midpoints (0.001 in.)		
	Deflection	Recovery	Residual	Deflection	Recovery	Residual
Dec. 1963	5	5	0	5	5	0
May 1965	4	7	-3	4	7	-3
Oct. 1965	7	7	0	6	6	0
June 1966	4	7	-3	4	7	-3
Nov. 1966	7	7	0	6	6	0
Nov. 1967	6	6	0	6	6	0
Nov. 1968	7	7	0	7	6	1
Oct. 1969	7	6	1	6	5	1
Nov. 1970	7	7	0	7	7	0

Figure 3. Average profile settlement.

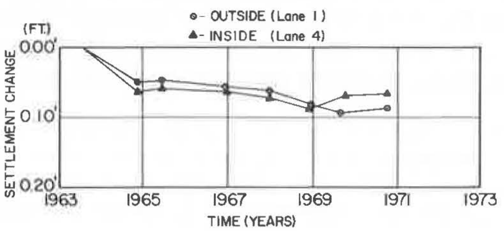

Figure 4. Typical cross section.

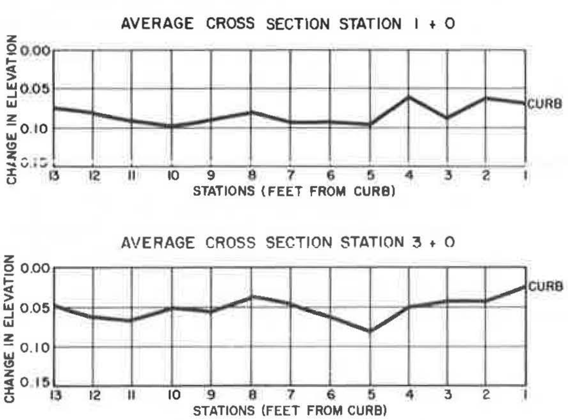

Table 4. Rut-depth measurements.

Date	Avg Value in Lane 4 (in.)		Avg Value in Lane 1 (in.)	
	Left Wheel-path	Right Wheel-path	Left Wheel-path	Right Wheel-path
Dec. 1966	0.45	0.15	0.30	0.20
Dec. 1967	0.40	0.15	0.30	0.20
Dec. 1968	0.45	0.45	0.20	0.25
Dec. 1969	0.45	0.20	0.20	0.20
Nov. 1970	0.40	0.20	0.25	0.35

In general, the composite pavement of the 3 approaches and the NJ-20 ramp discussed here appear to be in good condition.

CONCLUSION

The conclusions are based primarily on the performance of the westbound lanes of the east approach where complete data were collected. The composite pavement has functioned in accordance with design objectives with no interference with traffic for required maintenance or repairs. After 8 years of satisfactory service, the composite pavement in the westbound lanes of the east approach was subjected to more than 14.8 million 18-kip equivalent axle repetitions and gave no indication of progressive deterioration. No localized differential settlement was observed. A high load-carrying capacity was achieved as evidenced by the low deflection measurements.

The latest condition survey indicates no reflection cracking and no pavement deficiencies. Roughometer measurements and rutting measurements are within acceptable limits. Riding qualities of the pavement have not deteriorated to any significant extent.

The estimated service life of the 8-in. base composite pavement was calculated by the AASHO system for remaining service life of flexible pavement. Although the composite pavement is not a true flexible pavement, it exhibits many flexible pavement characteristics such as rutting and surface flexibility. A conservative estimate of the remaining service life of the composite pavement is 30 years or a total service life of at least 38 years. The cost of the 8-in. base composite pavement is approximately 15 percent greater than that of the reinforced portland cement concrete pavement. However, the increased expenditure produces approximately 23 percent greater service life.

The 6-in. plain concrete base pavement on the NJ-20 ramp also performed well. That ramp, which experiences comparatively light traffic, did not show patched areas or evidence of a need for maintenance.

ACKNOWLEDGMENTS

The able assistance of John Quinn, Kenneth Afferton, and William Van Breemen in the preparation of this report is gratefully acknowledged.

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DESIGN CONSIDERATIONS FOR RESURFACING PAVEMENTS WITH CONCRETE

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The most common methods for determining concrete resurfacing thickness are reviewed, and the major factors affecting the design of concrete resurfacing are discussed. It is suggested that limitation of slab deflection is of great importance. Deflection at joints, cracks, and free edges is greater than deflection at some distance from those discontinuities. Based on laboratory data, maximum slab deflection for various methods of load transfer across joints or cracks is proposed. The methods of load transfer discussed are aggregate-interlock, dowels, and continuous reinforcement. The effect of load position and method of load transfer on slab deflection is noted, and the structural benefit of tied concrete shoulders is indicated. Values for the slab support capacity of subgrades, subbases, and existing pavements are suggested, and, based on concrete pavement performance at the AASHO Road Test, maximum allowable slab deflection was calculated to be 0.025 in. Equating slab depths determined by calculation and field performance made it possible to establish a relation between static loads and truck traffic. Concrete resurfacing thickness was then related to truck traffic, method of load transfer across transverse joints or cracks, shoulder type, and slab support. A design example is used to illustrate how concrete resurfacing thickness may be determined. The design and performance of some recent concrete resurfacing projects are considered. The need for a stress relief or leveling course for concrete resurfacing of both concrete and bituminous pavement is discussed. The use of the PCA roadmeter to determine present serviceability index and its application to concrete pavement and resurfacing design are indicated.

•THE MOST common procedure for determining concrete resurfacing thickness on an existing concrete pavement is that developed by the Corps of Engineers (1). Resurfacing thickness is related to required thickness of a new pavement, thickness and condition of the existing pavement, and bond between the resurfacing and the existing pavement. That design method indicates that direct or partially bonded resurfacing may be thinner than separated or unbonded resurfacing. That does not appear to be justified based on the performance of highway resurfacing projects.

The most common methods for determining the slab thickness of concrete resurfacing on bituminous pavement, and also on new concrete pavement, are those of the Portland Cement Association (PCA) and the American Association of State Highway Officials (AASHO) (2, 3). In those methods, slab thickness is related to anticipated axle loads, concrete flexural strength, and slab support capacity of material under the slab. It should be noted that for the same design conditions the PCA and AASHO methods generally indicate that different slab thicknesses are required!

Because the PCA and AASHO design methods are not in agreement and the Corps of Engineers design procedure must be related to one of those methods, a discussion of factors affecting concrete pavement design seems appropriate.

Concrete resurfacing is a means of strengthening, restoring smoothness of ride on, and providing an appropriate surface texture to both concrete and bituminous pavements.

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

Pavements may be built initially with a concrete surface or be stage-constructed with concrete, i.e., be resurfaced with concrete. In both cases, the design of a concrete slab requires determination of maximum allowable slab deflection at joints, cracks, or free slab edges; load transfer method across joints or cracks; load position relative to joints, cracks, or free slab edges; support capability of the materials on which the slab will be placed; loads or traffic to which the slab will be subject; slab depth compatible with the above factors; joint or crack type and spacing; need for reinforcement between joints or cracks; and concrete quality.

The following discussion is an endeavor to indicate the relative influence of the above factors in the design of concrete resurfacing, which is, in reality, a concrete pavement having relatively good slab support.

SLAB DEFLECTION

Excessive slab deflection results in the type of distress often attributed to pumping, that is, faulting of joints and cracks and slab disintegration at the free edge of continuously reinforced concrete pavements. Deflection measurements indicate that the following equation, developed by Westergaard in 1926, adequately predicts interior slab deflection d_i:

$$d_i = P/8kl^2$$

where

P = load, lb;
k = subgrade modulus, pci;
$l^4 = Eh^3/[12(1 - \mu^2)k]$;
h = slab depth, in.;
μ = Poisson's ratio; and
E = elastic modulus of concrete, psi.

Values of l for a range of h and k values may be found in another report (4). Interior deflections may also be calculated by means of the PCA influence charts for single axles, tandem axles, and the like. In this discussion, load is considered to be a wheel load or half a single axle load. Interior deflections occur under loads applied approximately 5 ft from joints or free edges; however, all other conditions being equal, slab deflection at joints, cracks, and free edges is greater than at slab interiors. Figure 1 shows the approximate slab deflection if a given load is placed at various locations on a pavement (5, 6, 7, 8). The difference in joint deflection for the various methods of load transfer and shoulder type indicates that slab thickness can be varied with the method of load transfer and shoulder type. The difficulty of measuring slab deflection in the field because of slab warping or curling is recognized. However, if the relative deflection at the load points shown in Figure 1 for different slab depth, joint design, and loading is obtained at the same time on the same day, those data would be of use in analyzing field performance of various pavement designs.

LOAD TRANSFER METHOD

The method of load transfer across joints and cracks is a major factor affecting maximum slab deflection. It is assumed that load transfer across transverse joints or cracks is provided by either aggregate interlock, dowels, or reinforcement. Longitudinal joints may be tied or untied, doweled or undoweled. If a longitudinal joint is not tied or doweled, slab depth at the joint should be the same as if it were a free edge. Tied joints may be weakened plane, keyed, corrugated, or plain providing that an appropriate amount of steel is used across the joint. The nonpositive nature of load transfer across joints that are not tied or doweled should be recognized. However, concrete pavements with undoweled transverse joints can be designed so that joint faulting will not be excessive for a predetermined service life. It is assumed that the load transfer capability of doweled joints and continuous reinforcement is similar (Fig. 1 and Table 1). If continously reinforced concrete pavement (CRCP) is properly designed and constructed, there is evidence that continuous reinforcement is superior to dowels as a method of load transfer (9). The effect of various methods of load transfer on maximum slab deflection is given in Table 1 and shown in Figure 1.

EFFECT OF LOAD POSITION

The structural benefit of a concrete shoulder tied to the slab should be noted (Table 1 and Fig. 1). Laboratory data indicate that, if a load is at least 2 ft from a free edge, a significant reduction in free edge deflection results (5, 6, 7, 8). As an alternative to using a tied concrete shoulder, the lane width could be increased and corrugations placed in the outside 2 ft to discourage its use by traffic. A curb and gutter tied to the slab would produce the same result. The effect of load placement on concrete pavement behavior is also documented elsewhere (10). Concrete pavements are now usually built with a uniform slab depth, i.e., a rectangular cross section. That results in free edges being weaker than tied longitudinal joints. The economic as well as structural advantage of constructing slabs with a trapezoidal cross section or a thickened edge should be considered in locations where concrete shoulders are not appropriate.

SLAB SUPPORT

Concrete slabs may be placed on subbases, on existing pavements, or directly on subgrades. Subbases and existing pavement can provide a nonpumping, all-weather construction platform. If an all-weather construction platform is not considered mandatory, pumping may be prevented by limiting slab deflection. That is possible with or without a tied concrete shoulder or curb and gutter, provided that an appropriate slab depth is used.

Subbases and existing pavements also increase slab support, and that should be recognized in slab thickness determination (11, 12). In the PCA and AASHO methods for determining slab thickness, slab support is estimated in terms of the Westergaard modulus of subgrade reaction k. The choice of an appropriate k value requires some engineering judgment, for k values vary considerably with testing procedure and the time of the year when testing is done (13, 14). An extensive study of concrete pavement performance indicates that, for practical purposes, there are 2 subgrade categories (15):

1. Soils having an AASHO classification of A-1, 2, and 3, i.e., soils having good vertical drainage such as sand and gravel; and
2. Soils having an AASHO classification of A-4, 5, 6, and 7, i.e., soils having poor vertical drainage such as clay.

Subgrade k values of 50 and 150 are used for A-4, 5, 6, and 7 and A-1, 2, and 3 soils respectively to represent the subgrade in its weakest condition (2). Data given in Table 2 may be used to estimate the slab support k value of subbases and existing pavements composed of a variety of materials. Data given in Table 2 were developed from charts used by the California Division of Highways to determine the k value on top of unstabilized granular material and cement-treated aggregate subbase (12). Plate load tests on cement-treated subbase and bituminous concrete subbase indicate that cement-treated subbase has a significantly higher k value than bituminous concrete (16). However, in the absence of deflection measurements at the joints and cracks of concrete pavement placed on top of those materials, it is assumed that they have similar slab support capability. It is also assumed that portland cement concrete has a slab support capacity similar to that of cement-stabilized material and bituminous concrete. That is reasonable if the existing concrete pavement is structurally damaged, e.g., if it has excessive joint or crack faulting. In any event, the k value for design purposes should be chosen with due regard to the subgrade type, existing pavement design (thickness of structural components and load transfer method if concrete), existing pavement condition, and reason for resurfacing.

MAXIMUM ALLOWABLE SLAB DEFLECTION

Concrete pavement at the AASHO Road Test was subjected to known traffic, and the performance was documented in terms of present serviceability index (PSI), a measure of ride smoothness varying between 0 and 5. Concrete pavement sections at the AASHO Road Test had an initial PSI of approximately 4.5 and were considered to have failed when the PSI dropped to 1.5. The minimum subgrade k value at the AASHO Road Test was approximately 50 pci (AASHO A-6 soil), and granular subbase varied from 3 to 9 in. All transverse joints were doweled, and longitudinal joints were tied and keyed (13). The maximum calculated slab deflection, $3d_1$ (Table 1), was determined and related to PSI (Table 3). Data given in Table 3 indicate that very good performance resulted if the maximum calculated slab deflection did not exceed 0.025 in. Calculated slab deflection is generally greater than slab deflection measured in the field because of the effect of slab curling at joints, cracks, and free edges caused by continual daily temperature change from the bottom to the top of a concrete slab. A maximum allowable calculated slab deflection of 0.025 in. is used in this discussion.

LOADS AND TRAFFIC

After a maximum allowable slab deflection is chosen, it is possible to calculate slab thickness for a range of static loads. Static loads of 3,000 to 15,000 lb, representing single axle loads of 6,000 to 30,000 lb, are used. For practical application, slab thicknesses obtained by using static loads must be related to slab thickness requirement based on field performance under traffic.

An extensive study of the performance of concrete pavements having undoweled joints has been made by Brokaw (15). That study related pavement smoothness to heavy truck traffic, subgrade soil type, and pavement age for a range of slab depths. Based on slab depths determined from calculation (using a static load and assuming a slab support k value and a maximum allowable slab deflection) equated to required slab depths determined from field performance (PSI versus age relation), static loads were related to traffic (Table 4).

SLAB DEPTH RELATED TO TRAFFIC, LOAD TRANSFER, SHOULDER TYPE, AND SLAB SUPPORT

Data given in Table 5 are based on a maximum allowable calculated slab deflection of 0.025 in. They indicate the structural benefit of dowels, continuous reinforcement, concrete shoulders, and improved slab support. For a specific project, they show that several different concrete pavement designs are available and that the most appropriate one can be selected.

JOINT TYPE AND SPACING

The choice of joint type and spacing is of paramount importance if distress associated with joint movement such as spalling and pavement blowup is to be prevented. In general, if all transverse joints are contraction joints, the joint spacing should not exceed 30 ft; if all transverse joints are expansion joints, the joint spacing should not exceed 80 ft (17). Longitudinal joint spacing should generally not exceed 15 ft. If joint spacing and type are as suggested above and joints are doweled or tied, joint sealing is not required.

REINFORCEMENT

Slab reinforcement should be used in jointed pavement if the transverse joint spacing exceeds 12 ft (4-in. slab), 15 ft (5-in. slab), and 20 ft (6-in. or greater slab). In both jointed pavement and CRCP, overstressing of reinforcement must be prevented. Accordingly, dowels should not be excessively misaligned and should have a cladding such as Monel metal, stainless steel, or possibly plastic (17, 18). If CRCP will be subject to large slab temperature change, the use of "elastic joints" or an increase in the steel to concrete area ratio should be considered (19, 20).

Figure 1. Effect of load position and method of load transfer on slab deflection.

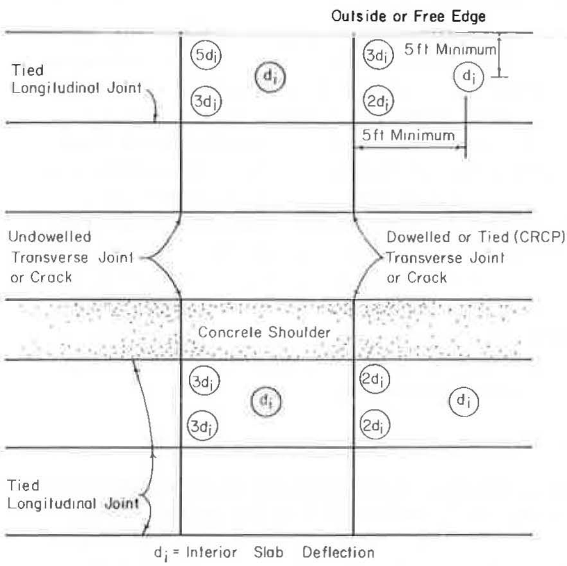

Table 1. Slab deflection related to transverse-joint load transfer method and shoulder type.

Load Transfer Method	Shoulder	Maximum Slab Deflection[a]
Aggregate interlock	Granular or bituminous material	$5d_i$
Aggregate interlock	Concrete[b]	$3d_i$
Dowels or continous reinforcement	Granular or bituminous material	$3d_i$
Dowels or continuous reinforcement	Concrete[b]	$2d_i$

[a]For location, see Figure 1.
[b]Concrete shoulders are same depth as slab at pavement edge. Longitudinal joints are tied. They may be weakened plane, keyed, corrugated, or plain.

Table 2. Slab support k values for 2 subgrade soils.

Material on Subgrade	Depth (in.)	Subgrade Soil: A-4, 5 6, and 7	Subgrade Soil: A-1, 2, and 3
	0	50	150
Granular base	6	75	200
Granular base	12 and more	125	250
Portland cement concrete, bituminous concrete, and cement-treated subbase[a]	4	125	250
	8 or more	250	500

[a]If there is 6-in. granular base beneath the 4-ft subbase, increase k by 50 pci; if 12 in. beneath, increase k by 100 pci.

Table 3. Maximum calculated slab deflection of concrete pavements at the AASHO Road Test.

Slab Depth (in.)	P[a] (lb)	k[b] (pci)	Least Number of Repetitions Carried by Failed Sections	Lowest PSI of Surviving Sections	Max Slab Deflection[c] (in.)
3.5	6,000	100	289,000	1.5-	0.059
5	9,000	100	291,000	1.5-	0.051
6.5	11,200	100	705,000	1.5-	0.043
8	15,000	100	768,000	3.4	0.043
3.5	3,000	50	1,114,000	3.7	0.042
5	6,000	75	725,000	3.1	0.040
6.5	9,000	100	1,114,000	1.8	0.035
5	6,000	100	1,114,000	3.3	0.034
3.5	3,000	75	1,114,000	4.0	0.034
9.5	15,000	100	1,114,000	2.2	0.033
8	11,200	100	1,111,000	4.6	0.032
11	15,000	75	1,114,000	4.0	0.031
8	9,000	75	1,114,000	3.9	0.030
9.5	11,200	75	1,114,000	3.7	0.028
6.5	6,000	75	1,114,000	4.1	0.027
2.5	1,000	50	1,114,000	4.2	0.023
6.5	6,000	100	1,114,000	4.2	0.023
2.5	1,000	75	1,114,000	4.4	0.019
9.5	9,000	75	1,114,000	4.5	0.017

[a]P = half single axle load.
[b]Subgrade k = 50 pci; subbase k estimated from Table 2.
[c]Max slab deflection = 3 d_i (see Table 1).

CONCRETE QUALITY

Concrete used in pavement must be adequately durable for the predicted service life. Concrete commonly used in pavements has a flexural strength of 500 to 700 psi (third-point loading at 28 days) or a compressive strength of approximately 3,500 to 4,500 psi at 28 days. In general, concrete of that quality has performed well, providing that air content was appropriate, the slab surface was not overfinished, and consolidation was adequate. However, particularly for heavily traveled pavement, the ability of high strength concrete (6,000 to 7,000 psi) to increase structural capacity and retain surface texture is worthy of future research (21).

DESIGN EXAMPLE

The subgrade is clay (AASHO A-6 soil), and the subbase is 6-in. granular material. The existing slab is 10-in. unreinforced slab with undoweled joints that have faulted excessively. The slab panels are uncracked (C = 1.0). k on top of the slab = 300 (Table 2). Average daily traffic on the design lane is 2,000 tractor semitrailers and combinations during a period of 30 years. The new pavement design is 9 in. of CRCP on 4-in. treated subbase. Resurfacing is to be CRCP with a stress relief course (unbonded). Resurfacing thickness by Corps of Engineers (1) method = $\sqrt{9^2 - 1.0 \times 10^2} = 0$. Resurfacing thickness by proposed method (Table 5) = 7 in. with bituminous shoulder or 5.5 in. with tied concrete shoulder.

RECENT RESURFACING PROJECTS

Since 1959, most of the highway resurfacing projects using concrete have been continuously reinforced, and most of those have been placed on a stress relief or leveling course (or both) of bituminous concrete. Those projects appear to be performing very well. During 1970-71, in Indiana and Georgia (Table 6), CRCP resurfacing was slip-formed directly over concrete pavement. The project in Indiana had 2 sections of 6-in. CRCP resurfacing, one placed directly on the existing slab and the other separated from it by a polyethylene bond breaker. The project in Georgia was placed directly on a concrete pavement and varied in thickness between 7 and 9 in. Also, CRCP resurfacing in Oregon and unreinforced concrete resurfacing in California were slip-formed directly over bituminous pavement without a leveling course.

The long-term performance of those projects will provide additional data on concrete resurfacing with and without a stress relief or leveling course. In general, a stress relief course is recommended prior to resurfacing with concrete. However, use of CRCP for resurfacing may allow the omission of a stress relief or leveling course, and that would reduce cost for a given pavement and shoulder depth and would minimize construction time.

Since the AASHO Road Test, relatively few data have been gathered on pavement PSI related to pavement age; and, as a result, meaningful documentation of pavement performance is lacking. After the development of the PCA roadmeter by Brokaw (22), a rapid method for measuring PSI became available, and engineers are now able to appraise with minimum effort the performance of pavements having a variety of designs. Before a pavement is resurfaced, its terminal PSI should be documented; after it is resurfaced, the new PSI should also be documented. By relating those data to traffic, slab support, and the like, one can determine performance and cost of pavement and develop realistic design methods.

SUMMARY

1. Existing methods for determining slab thickness of concrete pavement and resurfacing should be reappraised. A more logical design method is proposed.

2. In concrete pavement or resurfacing design, limitation of slab deflection is of great importance. Maximum slab deflection occurs where transverse joints or cracks intersect the outside or free edge of the pavement. If concrete shoulders are tied to the pavement, the free edge deflection is reduced.

Table 4. Relation between static loads and traffic.

Static Load (lb)	Slab Depth (in.) A-4, 5, 6, and 7 Subgrade[a]	Calculated k = 75[b]	A-1, 2, and 3 Subgrade	Calculated k = 200	ADTST[c] on Design Lane for 30 Years[d]
3,000	6	6.5	5	4.5	20
6,000	9.5	10	7.5	7	200
9,000	13	13	9.5	9.5	800
12,000	16	15.5	11	11	2,000
15,000	19	18	13	13	5,000

[a]Granular subbase was under all slabs. Maximum allowable slab deflection = 0.025 in. = 5d_i.
[b]k values were estimated from data given in Table 2.
[c]Average daily tractor semitrailer and combination traffic.
[d]Terminal PSI = approximately 2.5.

Table 5. Slab depth related to traffic, load transfer, shoulder type, and slab support for maximum calculated slab deflection of 0.025 in.

ADTST on Design Lane for 30 Years	Transverse Joint Load Transfer[a]	Shoulder Type[b]	Slab Depth[c] (in.) k = 50	k = 150	k = 500
5,000	A	G or B	20.5	14.5	9.5
	A	C	14.5	10	7
	D or CR	G or B	14.5	10	7
	D or CR	C	11	8	5.5
2,000	A	G or B	17.5	12.5	8.5
	A	C	12.5	9	6
	D or CR	G or B	12.5	9	6
	D or CR	C	9.5	7	4.5
800	A	G or B	14.5	10	7
	A	C	10.5	7.5	5
	D or CR	G or B	10.5	7.5	5
	D or CR	C	8	6	4
200	A	G or B	11	8	5.5
	A	C	8	6	4
	D or CR	G or B	8	6	4
	D or CR	C	6	4.5	3

[a]A = aggregate interlock; D = dowels; and CR = continuous reinforcement.
[b]G = granular material; C = concrete; and B = bituminous material. Concrete shoulders are same depth as slab at pavement edge. Longitudinal joints are tied.
[c]Slab depth is that required at free edge or at longitudinal joint when adequately tied concrete shoulders are used. Slab depth must be sufficient to provide adequate cover for dowels or reinforcement.

Table 6. Highway projects for concrete resurfacing.

State	Route	Approximate Area (yd²)	Year Built	Existing Concrete Slab (in.)	Resurfacing (in.)	Continuous Reinforcement	Paving Method
Texas	I-35	15,000	1959	7 + B[a]	7	0.56	Form
	I-35	120,000	1965	9	6 + B[c]	0.57	Slip form
Illinois	I-70	10,000	1967	10 + B	6 + B	0.7 and 1.0	Form
	I-70	10,000	1967	10 + B	7 + B	0.7 and 1.0	Form
	I-70	50,000	1967	10 + B	8 + B	0.6	Form
California	I-80	28,000	1968	8	8 + V[d]	0[f]	Slip form
	US-99	15,000	1968	4 + B	8	0	Slip form
	I-8	40,000	1969	8	6 + B	0	Slip form
	US-99	100,000	1971	Bituminous[b]	8½	0	Slip form
Indiana	I-69	15,000	1970	9	6	0.6	Slip form
	I-69	40,000	1971	9	6 + P[e]	0.6	Slip form
Arkansas	I-55	24,000	1971	9	6 + B	0.6	Form
Mississippi	I-20	30,000	1971	9	6 + B	0.6	Slip form
Georgia	I-75	45,000	1971	8	7	0.7	Slip form
	I-75	146,000	1971	8	8	0.6	Slip form
Oregon	I-80	66,000	1971	Bituminous	7 to 9	0.6	Slip form
Maryland	I-70	130,000	1972	9	6 + B	0.6	Form

[a]Original concrete slab was previously resurfaced with bituminous concrete.
[b]Original pavement was not concrete.
[c]A bituminous concrete layer was placed prior to concrete resurfacing.
[d]Various bond breakers were used.
[e]A polyethylene bond breaker was used.
[f]Joints were undoweled, and concrete resurfacing was unreinforced.

3. If deflection at the interior of a slab is d_i, the maximum slab deflection is approximately $5d_i$ for a pavement with undoweled joints and $3d_i$ for a pavement with doweled joints or reinforcement across cracks. The use of a tied concrete shoulder reduces the maximum slab deflection to $3d_i$ and $2d_i$ respectively. The maximum allowable slab deflection is calculated to be 0.025 in.

4. For design purposes, subgrade k values of 50 and 150 are appropriate for AASHO A-4, 5, 6, and 7 and A-1, 2, and 3 soils respectively. Subbases and existing bituminous or concrete pavements improve the slab support k value, and that improvement should be considered in the design process.

5. The calculated slab depth based on a maximum slab deflection is equated to the slab depth of pavements in service, and a relation between static loads and truck traffic is established.

6. Concrete pavement or resurfacing need not have a uniform depth cross section. Slab depth should be varied with due regard to truck traffic on design lane, load transfer method across joints, shoulder type, and slab support of existing pavement.

7. Concrete resurfacing projects in service indicate that a stress relief or leveling course is generally desirable prior to resurfacing concrete pavement, it may be possible to omit a stress relief or leveling course if the concrete resurfacing is continuously reinforced, and concrete may be slip-formed directly over bituminous pavement without a stress relief or leveling course.

8. The present serviceability index of existing pavement and resurfacing should be determined by means of the PCA roadmeter so that with a minimum of effort the true performance and cost can be determined and realistic design methods developed.

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COMPOSITE PAVEMENT EXPERIMENTS IN ROMANIA

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•TO ADD to the information on the performance of composite pavements and overlays, I would like to report on 2 interesting experiments with composite pavements that were done in Romania several years ago.

One of the experiments was organized and led to completion by Laurentiu Nocoara, of the Polytechnic Institute of Timisoara. The structural design of the pavement was made by the author.

The structure of the pavement (from the top to the bottom) was as follows:

Course	Amount
Asphalt concrete surface, cm	7.5 (3 in.)
Crushed-stone base, cm	10.0 (4 in.)
Cement-treated subbase, cm	25.0 (10 in.)
Sand insulation against frost penetration, cm	15.0 to 20.0 (6 to 8 in.)

The surface of the crushed-stone base course was leveled with sand asphalt to retain the stability of the layer. The subbase was made of uniformly graded sand with 8 percent portland cement. The subgrade was a clayey sand (E = 350 kgf/cm^2 or E = 4,900 psi).

The expected traffic was 4.5 million or 10.0 metric tons (about 22.0 kips) equivalent axles.

After 3 years of heavy traffic, no cracks were observed at the surface of the pavement, which means that the cracks from the cement-treated base were not transmitted to the top layer. No signs of fatigue appeared although more than 1 million equivalent axles had passed on the highway.

The second experiment was conceived, organized, and lead to completion by Mircea Velica, also of the Polytechnic Institute of Timisoara.

The structure of the pavement (from the top to the bottom) was as follows:

Course	Amount
Asphalt concrete surface, cm	7.5 (3 in.)
Cement concrete base, cm	20.0 to 25.0 (8 to 10 in.)
Sand insulation against frost penetration, cm	10.0 to 20.0 (4 to 8 in.)

The cement concrete base course had no dowels or any kind of bond between the 4-m (13 ft) slabs. The subgrade was either sandy clay (E = 300 kgf/cm^2 or 4,200 psi) or silty sand (E = 350 to 400 kgf/cm^2 or 4,900 to 5,000 psi).

The joints of the cement concrete slabs were covered with bituminous cardboard (having a width equal to the thickness of the slab on both sides of the joint, without being fastened to the cement concrete slabs) before the asphalt concrete surface course was laid.

After more than 4 years of heavy traffic, no cracks were observed at the surface of the pavement.

Based on these experiments and those discussed in the preceding papers, it should be recognized that composite pavements can be used with great success in highway construction.

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PREDICTION OF REFLECTION CRACKING IN PAVEMENT OVERLAYS

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This paper deals with the problem of predicting the rate at which cracks in an underlying layer of a pavement will reflect through a bituminous overlay. It is an application of the principles of fracture mechanics to the problem of the initiation and growth of reflection cracks under the combined influence of repeated vehicular loading and changes in temperature. Both the theory of linear elastic fracture mechanics and that of delayed fracture in viscoelastic materials are used to formulate a method of solution. A brief discussion of the basic concepts, principles, and mathematical formulation is given. Finally, a few simplifying assumptions that lessen the amount of numerical computations required are made, and the applicability of the theory is illustrated by an example. Certain conclusions are drawn, and the need for future research in this area is outlined.

•THE PRESENT state of the art of designing overlays for pavements is to a large degree based on experience gained by trial and error and empirical relations developed from in-service highways and airport runways. As long as the designer is dealing with foundation soils, material properties, environmental factors, traffic loading, and construction techniques that are similar to those from which the relations were developed, the performance can be reasonably well predicted. However, because that is not usually the case, a need exists for a more rational approach to pavement design and pavement rehabilitation.

The present methods of designing overlays are based on some tolerable level of deflection (1, 2); those for overlays on portland cement concrete are based on the stress in the concrete slab at the base (1). If the pavement on which the overlay is to be constructed is cracked or jointed, there will be a substantial amount of reflection cracking through the overlay. Such reflection cracking can ruin the performance of an otherwise good overlay, for it will permit the entry of water through the cracks, leading to softening of the subgrade, pumping, and transverse faulting. Thus, reflection cracking can indeed control the design of an overlay.

Existing methods of analysis and design of overlays do not consider the problem of reflection cracking directly. However, certain construction techniques, such as breaking up the cracked or jointed pavement slabs into small pieces or small blocks, are used to minimize or delay the occurrence of reflection cracking. The disadvantages of those techniques are, of course, the tremendous loss in load-carrying capacity, caused by breaking up the pavement slabs, and the costs involved. A better solution is required—one in which the inherent strength of the slab can be used as much as possible. It is believed that the development of a basic theory that considers the mechanism of the formation and development of the reflection cracking and that describes the processes involved in terms of the invariant material properties of the pavement system, the traffic, and the environment is the best approach to finding a better practical solution to the problem.

The development of a rational method of overlay design not only will enable the prediction of the service lives of pavements carrying more modern aircrafts and

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automobiles with higher wheel loads, wider tires, and different configurations but also will provide a basis for evaluating the advantages of using new materials, new construction techniques, new types of equipment, and better quality control. Such a design method will remove the factors inhibiting innovative design and lead to superior design concepts and pavement rehabilitation techniques.

BASIC THEORETICAL APPROACH

Overlays may develop all types of distress: rutting, disintegration, and cracking. For rigid or flexible overlays on concrete pavements, reflection cracking is the most prevalent form of distress.

The methodology for the analysis of cracking in flexible pavements due to the effects of repeated vehicular wheel loading has been presented in another report (3). It is assumed that a "starter" crack c_0 initiates (i.e., begins to grow) from the first loading cycle from inherent flaws in the material. The rate of crack propagation dc/dN is proportional to the fourth power of the stress-intensity factor K, which measures the general transmission of load to the vicinity of the crack tip in accordance with the loading, geometrical, and boundary conditions (4):

$$\frac{dc}{dN} = A[(\Delta K)^4 - K_0^4] \quad (1)$$

where

c = crack length,
N = number of cycles,
A = materials property,
K_0 = endurance limit ($\simeq 0$ for asphaltic concrete), and
ΔK = increment in K due to the passage of a wheel load.

As the crack continues to grow, the value of K increases correspondingly until K reaches its critical value K_c, when rapid crack propagation occurs. K_c is a property of the material and is a function of the temperature and rate of loading.

For reflection cracking in an overlay, the same principles apply. The discontinuity in the underlying pavement slab produces bending and shear stress concentration at inherent flaws in the overlay along the line of the crack or joint. That causes nucleation of the flaws into microscopic starter cracks. The cracks grow under the influence of both traffic loading and the stresses produced by temperature and moisture changes.

The analysis of crack propagation is considerably simplified by the fact that the line of cracking is already known. In general, the crack will propagate in 2 modes: the opening mode and the in-plane sliding mode (Fig. 1). Usually the opening mode is much more important. The problem is, therefore, reduced to a simple application of fracture mechanics. Once the range and distribution of the starter flaws in the overlay along the potential line of cracking and the properties of the component layers of the pavement as outlined in an earlier report (3) have been determined experimentally, the rate of crack propagation can be obtained from the crack growth law:

$$\frac{dc}{dN} \; A_1(\Delta K_1)^4 + A_2(K_2)^4 \quad (2)$$

where K_1 and K_2 are the stress-intensity factors for the 2 modes of cracking, under the stresses produced by traffic loading. Equation 2 is applicable when the mean load is constant. For a variable mean load, Roberts and Erodogan (5) give the following formula:

$$\frac{dc}{dN} \; A'(1 + \beta)^2(\Delta K)^4 \quad (3)$$

for $K < K_c$, where $\beta = (K_{max} + K_{min})/2\Delta K$.

In general the overlay will be subjected to 2 types of loading: (a) repeated traffic loading, which produces pulses of short duration, and (b) stresses due to changes in temperature, which are usually sustained over long periods.

The response of the pavement due to the passage of wheel loads may be considered to be elastic if the speed of the vehicle is greater than about 15 mph, so that linear elastic fracture mechanics is applicable and the crack growth will follow Eq. 3. However, the response of the pavement due to stresses produced by slowly varying temperatures is essentially viscoelastic in nature, and therefore a viscoelastic analysis is needed to obtain the rate of crack propagation. Such an analysis has been presented by Wnuk (6) and is described briefly below.

According to Wnuk, a crack in a viscoelastic material initiates after a time

$$t_* = \Psi^{-1}(n) \tag{4}$$

where

$n = (K_{1c}/K_1)^2$,
$\Psi(t)$ = compliance function $= J(t)/J(0)$,
$J(t)$ = compliance at time t, and
$\Psi^{-1}(t)$ = inverse compliance function.

The crack then grows at a finite rate until eventually at time t_{**}, when the stress-intensity factor reaches its critical value K_{1c}, catastrophic failure occurs. The rate at which the subcritical crack grows is given by

$$\Psi(\Delta/\dot{c}) = (K_{1c}/K_1)^2 \tag{5}$$

for $t_* < t < t_{**}$, where

$$\Delta = \frac{\pi}{8(1-\nu^2)}(K_1/\sigma_y)^2, \text{ and}$$

$$\dot{c} = \frac{dc}{dt} = \text{rate of crack growth.}$$

Equation 5 may be rewritten as

$$\dot{c} = \Delta/\Psi^{-1}(K_{1c}/K_1)^2 \tag{6}$$

If the wheel loads pass over the crack at regular intervals at the rate of n per day, i.e., N = nt, then the total rate of crack growth $\dot{c}_t$ due to both vehicular loading and temperature stresses will be given by

$$c_t = nA_1'(1+\beta)^2(\Delta K_1)^4 + nA_2'(1+\beta)^2(\Delta K_2)^4 + \Delta/\Psi^{-1}(K_{1c}/K_1)^2 \tag{7}$$

for $t_* < t < t_{**}$, and

$$\dot{c}_t = nA_1'(1+\beta)^2(\Delta K_1)^4 + nA_2'(1+\beta)^2(\Delta K_2)^4 \tag{8}$$

for $t < t_*$.

Equations 7 and 8 are sufficient to completely describe the rate of growth of the reflection crack at any time during the loading history.

The stress-intensity factor K is the dominant parameter controlling the rate of crack growth. Unfortunately, the K value of a crack in an overlay over a joint in a concrete slab with dowel bars or steel reinforcement bars across the joint is difficult to determine and requires a 3-dimensional finite-element analysis. Such computer programs are available but are rather expensive to operate. Perhaps in the near future some simpler numerical method for obtaining the stress distribution for 3-dimensional crack problems may become available.

However, in order to illustrate the potential applicability of the method, a number of simplifications will be introduced to minimize the amount of numerical computation.

ILLUSTRATIVE EXAMPLE

Consider a 2-dimensional simplification of a pavement system consisting of a 3-in. asphaltic concrete overlay on an 8-in. jointed concrete slab without load transfer dowels, as shown in Figure 2. Under the influence of vehicular loading, an inherent starter crack in the asphaltic overlay directly above the joint in the concrete slab begins to grow with the first loading cycle and propagates in accordance with the crack growth laws previously discussed. Assume also that the asphalt concrete overlay was constructed on a day when the temperature was 110 F, and at that time the joint in the concrete slab was completely closed. If the temperature drops to 60 F, say, a tensile stress will be induced in the overlay because of the contraction of the pavement layers between the contraction joints. Under that sustained stress, the crack grows continuously but not before an initiation period $t_* = \Psi^{-1}(n)$.

Experience has shown that the resistance to cracking in the in-plane sliding mode K_2 is generally much greater than the resistance in the opening mode K_1. As a further simplification, mainly because of the lack of sufficient experimental data, the crack is considered to propagate only in the opening mode.

Based on experimental data, the following material properties are assumed:

Property	Value
Asphalt concrete	
Complex modulus E_1, psi	250,000
Poisson's ratio	0.4
Critical stress-intensity factor K_{1c}, lb-in.$^{-3/2}$	400
Yield stress in tension σ_y, psi	250
Thermal coefficient of expansion, in./in./F (low)	7.0×10^{-6}
Crack propagation constant A_1'	1.0×10^{-13}
Concrete	
Young's modulus E_2, psi	3×10^6
Poisson's ratio	0.3
Thermal coefficient of expansion, in./in./F	7.0×10^{-6}
Subgrade	
Young's modulus E_3, psi	4,500
Poisson's ratio	0.4

In addition, the asphalt is assumed to behave as a standard linear solid as represented by the model shown in Figure 3. The creep compliance function $\Psi(t)$ is, therefore, given by

$$\Psi(t) = 1 + \frac{E'}{E''}\left(1 - e^{-(E''/\lambda)t}\right) = 1 + 2\left(1 - e^{-2.0 \times 10^{-4} t}\right) \qquad (9)$$

Stress-Intensity Factor K Due to Vehicular Loading

The K_1 value was calculated as a function of the crack depth c for the vehicular loading shown in Figure 2; a 2-dimensional finite-element program was used. The graph of K_1 versus c is shown in Figure 4. The shape of the K_1 versus c curve is rather unusual; the value of K_1 decreases as crack length increases. That is because the modulus of the concrete slab is very much greater than the modulus of the asphalt concrete overlay, so that the behavior of the composite pavement is controlled to a large extent by the concrete slab.

Stress-Intensity Factor K_1 Due to Temperature Drop

The tensile stress σ in the asphalt concrete section (α for asphalt concrete and concrete are assumed to be equal) is given by

$$\sigma = \alpha E \Delta T \qquad (10)$$

where

E = long-term viscoelastic modulus, and

ΔT = drop in temperature of 50 F (assumed for duration of 300 days).

Therefore, $\sigma = 7.0 \times 10^{-6} \times \dfrac{250{,}000}{3} \times 50$ psi $= 37.2$ psi.

The value of K_1 for that state of stress is given in another report (7) and is shown as a function of the crack length c in Figure 4.

The value of K_1 versus c for the combined loading is also shown in Figure 4.

Elastic Crack Growth Rate Due to Vehicular Loading and Temperature Stresses

The crack propagation law is

$$\frac{dc}{dt} = nA_1'(1+\beta)^2(\Delta K_1)^4 \tag{11}$$

where

$A_1' = 1.0 \times 10^{-13}$,

$\beta = (K_{1\,max} + K_{1\,min})/2\Delta K_1$, and

t = time in days.

If the vehicular traffic consists of 1 million equivalent 18-kip axle loads in 20 years, then the value of $n = N/t = 137$ axle loads per day. Thus, all the quantities in Eq. 11 are known, and the rate of crack propagation can be calculated. The results are shown in Figure 5. Evidently the rate of crack growth is considerably increased by the presence of the temperature stresses.

Viscoelastic Crack Growth Rate Due to Temperature Stresses

From Eq. 6,

$$\dot{c} = \Delta/\Psi^{-1}(K_{1c}/K_1)^2 \tag{12}$$

for $t_* < t < t_{**}$.

The graphs of $\Psi(t)$ and $\Psi^{-1}(n)$ are shown in Figure 3. Thus, the values of $\dot{c}$ as a function of the actual crack length can be easily obtained. That is shown in Figure 5.

The initiation time for viscoelastic crack growth is given by $t_* = \Psi^{-1}(K_{1c}/K_1)^2$, where K_1 corresponds to the initial crack length. The graph of t_* versus c is shown in Figure 6. Evidently initiation occurs when the crack length is approximately 1.55 in.

Reflection Crack Growth as a Function of Time

Assuming that the "starter flaw" in asphalt concrete $\simeq 0.05$ in., the actual growth of the reflection crack due to the combined effects of vehicular loading and temperature change can now be obtained. That is shown in Figure 7. The reflection crack will propagate completely through the asphalt concrete overlay in about 280 days.

CONCLUSION

A method of predicting the rate of growth of reflection cracking in overlays has been presented. The method is quite general and permits the evaluation of the relative and combined effects on the growth of reflection cracking of both vehicular loading and temperature-induced stresses.

Certain simplifying assumptions were made in order to present an illustrative example. Because some of the assumptions were oversimplified, it is not appropriate to make any firm conclusions from the results. Nevertheless, it appears that the

Figure 1. Modes of deformation of reflection crack.

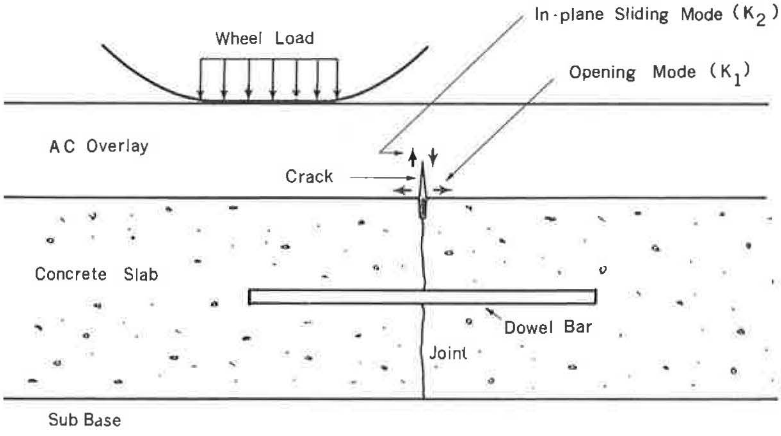


Figure 2. Two-dimensional simplification of problem.

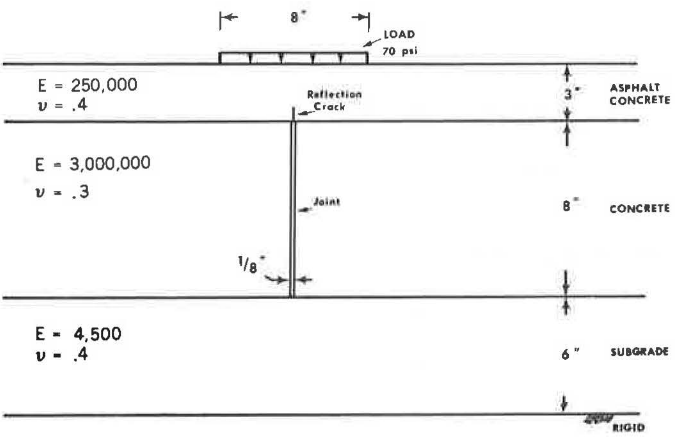


Figure 3. Asphalt concrete model and graphs of $\psi(t)$ and $\psi^{-1}(n)$.

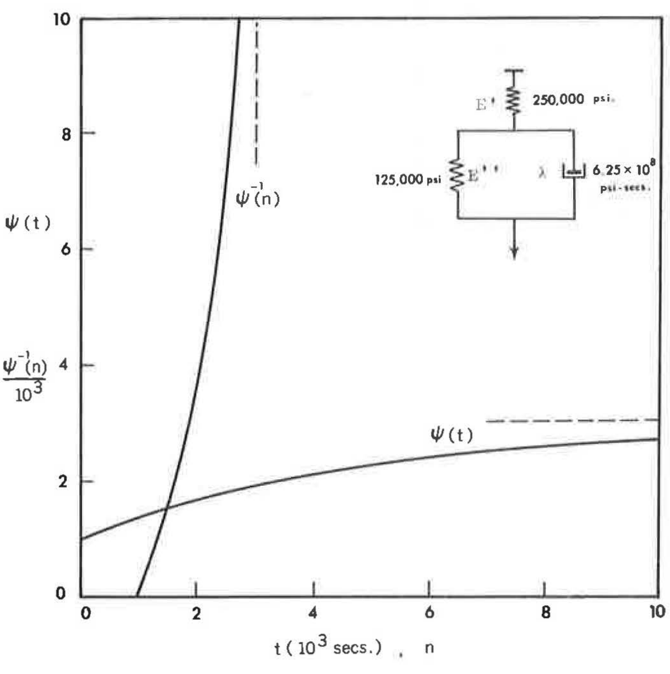


Figure 4. K_1 for vehicular loading and temperature stresses.

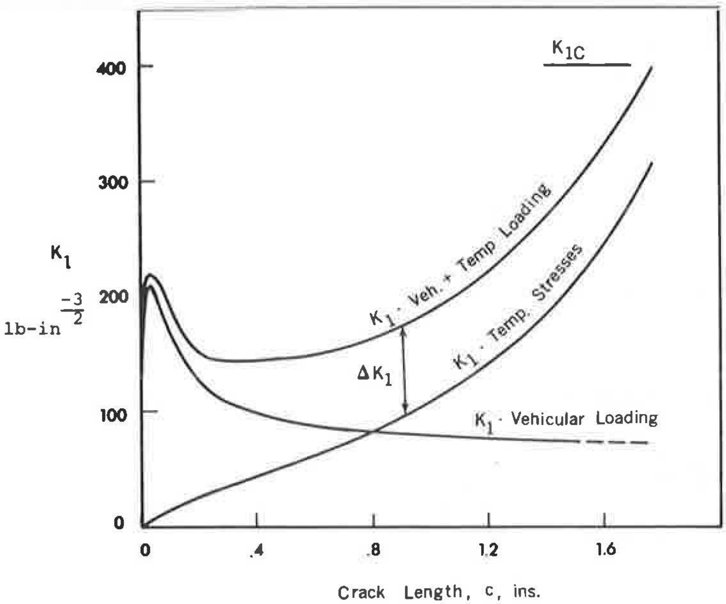


Figure 5. Growth rate of elastic and viscoelastic reflection crack.

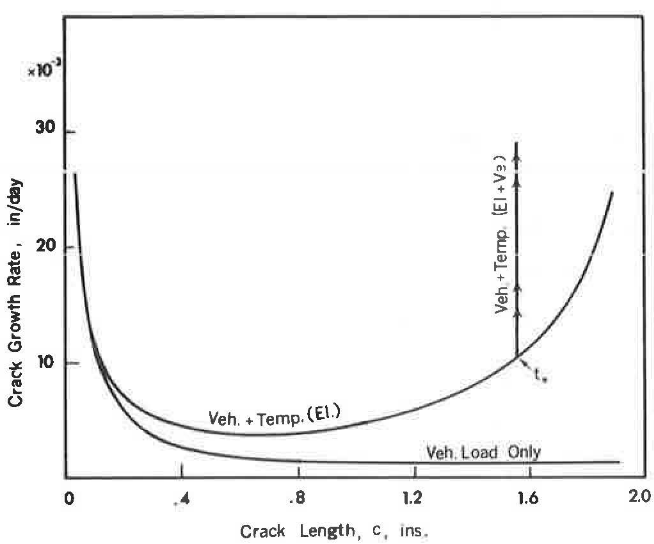


Figure 6. Initiation time for viscoelastic crack growth.

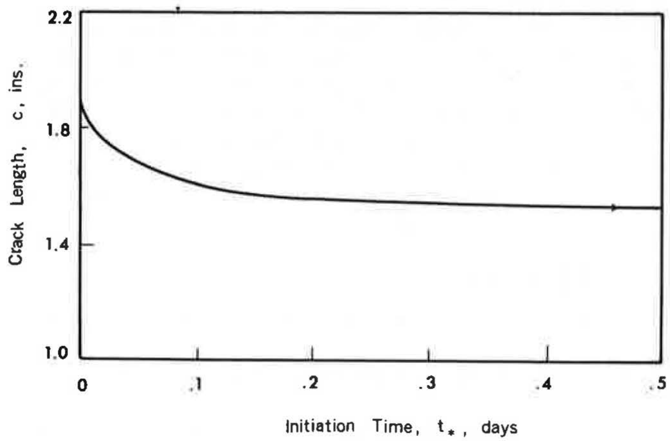
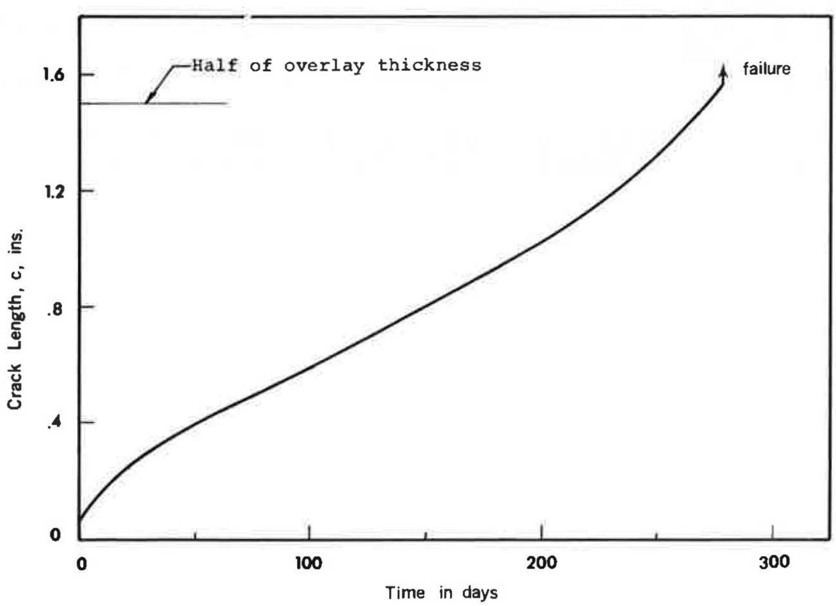


Figure 7. Growth of reflection crack under vehicular loading and temperature stresses.



method yields realistic results and that the effect of temperature change on the growth of the reflection cracking is rather substantial.

In this paper, it has been assumed that the resistance to cracking in the in-plane sliding mode is very much greater than that in the opening mode, and the former has been neglected. That assumption may not be true and should be checked experimentally. It is also recommended that the method be completely verified experimentally. The validity of this method for predicting reflection cracking means that a powerful tool is available that will make the task of minimizing the occurrence of reflection cracking much simpler, will remove those factors inhibiting innovative design, and will thus lead to superior design concepts and pavement rehabilitation techniques.

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WHAT WE HAVE LEARNED TO DATE FROM EXPERIMENTAL CONCRETE SHOULDER PROJECTS

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Concrete shoulders have been in use on urban expressways in some areas for years. They are now coming into use for major rural highways as well. Several experimental projects, most of them initiated under the FHWA National Experimental and Evaluation Program, have been built in recent years, and more are under construction or planned. The paper summarizes performance to date, examines design details developed from experimental projects, and makes recommendations for design for maximum safety and economy. The paper suggests that further study should be given to (a) effects on temperature and moisture conditions within the pavement system as the result of a light-colored shoulder surface and (b) effect on structural capacity of a concrete roadway pavement when a tied and keyed concrete shoulder is added.

•CONCRETE shoulders have been used for years in urban areas and have functioned primarily to furnish additional emergency parking or travel area and to improve surface drainage facilities. In general, the same thickness has been used for the shoulder as for the main roadway. Tie bars have been used to hold the shoulder to the roadway slab, and generally no sealant has been used in the joint where the slabs join.

The performance, low maintenance, and simplicity of construction of these sections have led to the construction of several experimental concrete shoulder projects on major rural highways in recent years. Three of these projects were in Illinois during the 1965-67 period. In 1970, the Federal Highway Administration became interested in the safety and economic aspects of concrete shoulders. A circular memorandum issued early that year urged each FHWA region to initiate concrete shoulder projects under the National Experimental and Evaluation Program (1). As a result of that program and of individually initiated state activities, projects were built in 6 states in 1971. The states that have projects constructed, under way, or planned are listed in Table 1.

As a result of experimental work, several states now include concrete shoulders as a standard shoulder design; details of concrete shoulder standards may be obtained from Illinois and Pennsylvania.

SUMMARY OF PERFORMANCE

Experimental concrete shoulder projects under observation since as early as 1965 (2, 3, 4) have shown the following general performance trends.

Thickness

From a structural standpoint, the 6-in. thickness used on projects to date has shown excellent performance. The oldest experiment, with an average daily traffic of 14,000 vehicles, has 6-in. shoulders built on in-place soil, which in some areas is a low bearing-value clay. After 7 years there is no evidence of structural failure. The same is true on later experimental sections.

Sponsored by Committee on Shoulder Design.

Where construction expediency has been a consideration, thickness of shoulder has been made the same as that of the main-line pavement. For the same reason, several projects have used a wedge-shaped shoulder section, meeting the roadway pavement thickness at its edge and thinning to 6 in. at the outer shoulder edge.

Subgrade

There has been little evidence from experimental projects that a subbase of any kind has contributed to concrete shoulder performance. Three early Illinois projects compared shoulder sections on in-place soil with those on subbases of various types and gradations. The small reduction in cracking that resulted where a subbase was used appears to be an insignificant advantage because shoulders carry no sustained traffic, and the cracks are subject to little traffic-induced deterioration.

Longitudinal Joint Sealing

The need for sealing the critical longitudinal joint between roadway and shoulder was investigated in an early shoulder experiment in Illinois (2). A $\frac{1}{2}$-in. × $\frac{3}{4}$-in. reservoir filled with conventional rubber-asphalt sealant was compared with an unsealed joint that had been edged only. After 5 years, both joints appear to be functioning properly, and there is no evidence of water leakage in the unsealed joint (Figs. 1 and 2). As a result of that experiment, current Illinois standards require edging of the longitudinal joint only and no sealing. That lead has been followed by at least one other state.

Although the need for sealing the longitudinal joint even in frost areas may be questioned in view of the Illinois experience, some states have felt that sealing, because of its relatively low cost, is additional insurance against leakage of water to the subgrade at this critical point. Where sealing has been specified, a low-cost rubber asphalt compound with a low extension-compression range has been used successfully as a sealant. For a sealant reservoir, dimensions not exceeding $\frac{3}{8}$ × $\frac{3}{4}$ in. appear to be sufficient.

Longitudinal Joint Key

The use of a formed key at the longitudinal joint in early experiments did not significantly improve shoulder performance. Keys were omitted from some later experimental projects. No evidence of vertical displacement of the shoulder as a result of that design change is apparent in the oldest of those projects after 5 years. The current Illinois and Pennsylvania standard shoulder designs do not include a key at the longitudinal joint.

Where the intent is only to keep the adjacent shoulder at the same level as the roadway pavement, the key can be omitted because tie bars and normal interlock have been shown to act as a sufficient restraint to faulting of the shoulder. However, advantages in improved joint effectiveness and increased structural capacity can be gained from having the pavement and the shoulder slabs work together. That is discussed in a later section on the effect on pavement structural capacity of a tied and keyed rigid shoulder.

Tie Bars

The need for deformed tie bars or tie bolts between roadway and shoulder slabs has been definitely established in experimental projects to date. Although it is true that some experiments have shown good performance without them, those instances are felt to be short term. During the life of the shoulder, the low relative cost of tie steel and the advantage of positive prevention of separation at the critical joint between the 2 slabs make its use a wise investment.

Most tie-bar designs have used No. 4 or No. 5 bars 30 in. long spaced at 30 in. That is the same design as used in the center joint of major highway pavements. It is possible that the amount of such tie steel can be reduced below that normally used between 12-ft roadway lanes because shoulders are in all cases narrower and generally

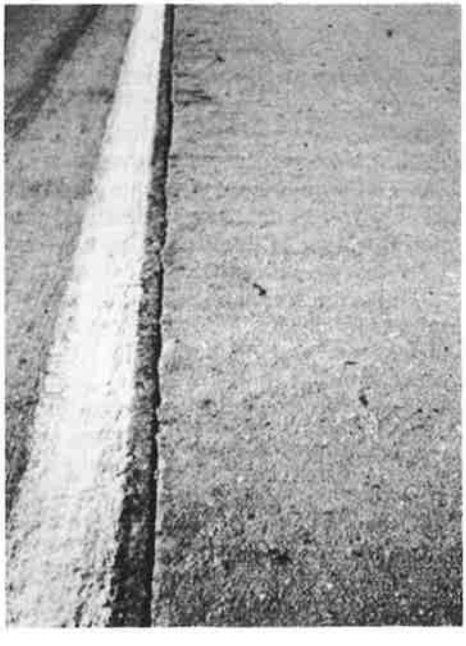
Table 1. Concrete shoulder projects.

State	Year	Route	Length (miles)	Width (ft)	Thickness (in.)	Jointing (ft)	Remarks
Built							
Ill.	1965	SR-116	5	8 and 4	6 P	50 and 100	Described in Refs. 2 and 3
	1966	I-74	4,400[a]	10 and 4	6 P	10 to 100	Described in Refs. 2 and 3
	1967	I-80	1.9	10 and 4	8 to 6 P	20	Described in Refs. 2 and 3
	1971	SR-72	7.5	10 and 4	6 P	20	
	1972	E-W Tollway	69	11 and 5	$8\frac{1}{2}$ to 6	Random	To be completed in 1973
Iowa	1971	I-80	1.25	10 and 6	6 P	20	Connected with reconstruction project
Md.	1969		3.6	3 to 8	7 R	40	
Mich.	1971	I-69	1.5	9	9 to 6 P	$17\frac{3}{4}$	
Neb.	1970-71	Neb-36	10	8	$5\frac{1}{2}$ P	15	
N. C.	1972	US-52	4.5		7	30	2-in. red concrete topping
N. D.	1972	I-29	14.7	10 and 3	8	CRCP	3-ft width slip-formed with roadway pavement
Penn.	1971	I-81	5.9	10 and 4	6 P	$15\frac{1}{2}$	
Texas	1971	I-30	6.9	10	8	CRCP	Experimental textures
Awarded							
Ala.		I-59	5	10 and 4	8		Evaluate designs and delineation methods
Ark.		I-430		10 and 4	8	CRCP	
Ky.		US-31W	3.7	10 and 4	5, 6, 7	20 P, 50 R	
N. M.		I-40	4.3	10 and 4	8	13, 19, 18, 12	Stage construction
Penn.		US-220	23,000[b]		6 P	$15\frac{1}{2}$	
			21,000[b]		8 to 6 P	$15\frac{1}{2}$	
			2,500[b]		8 P	$15\frac{1}{2}$	
W. Va.		I-77	30,200[b]		8	20	
		I-64	14,300[b]		8	20	
Planned							
Ariz.		I-10		10 and 5		Skewed and random	
Calif.		Calif-101					
Ga.		I-75	8	10	11 to 6		No subbase
Idaho		I-90					
Minn.				10 and 3			
Nev.		I-15	2.2	10 and 4			
Ohio		I-675	4.2	10			
S. D.		I-29					
Utah		I-15	1		9	Random	

[a]Feet. [b]Square yards.

Figure 1. Sealed joint on I-80 in Illinois after 5 years of service.

Figure 2. Unsealed joint on I-80 in Illinois after 5 years of service.



thinner and undergo little stress from loads. Design formulas (5) show that tie-bar requirements for 6-in. thick shoulders are as follows:

Shoulder Width (ft)	Tie-Bar Dimension (in.)	Spacing (in.)
10	$\frac{3}{8} \times 15$	30
	$\frac{1}{2} \times 20$	52
4	$\frac{3}{8} \times 15$	72
	$\frac{1}{2} \times 20$	132

Performance of such reduced designs should be checked experimentally. If there are cost savings to be realized from such reductions and if performance is not sacrificed, tie-bar requirements for concrete shoulders could be changed accordingly.

Contraction Joints

The effectiveness of transverse contraction joints at short spacings in reducing cracking in shoulder experiments has been demonstrated. A 20-ft spacing has been used in most shoulder projects. Plain concrete shoulders with joints at 20-ft spacing have been used adjacent to continuously reinforced concrete pavement whose crack interval averaged about 4 ft. Observations of 2 shoulder projects 5 and 6 years old show no reflective cracking in the shoulder from the numerous cracks in the adjacent CRC pavement.

There is no indication that contraction joints need to be uniformly spaced or that 20 ft should be a minimum spacing. In a major tollway project recently awarded, the same random spacing pattern was specified in the concrete shoulders as in the mainline pavement. The contractor was given the choice of skewing the shoulder joints to match the skew in the roadway pavement joints or of placing them at right angles to centerline.

Contraction joints in shoulder projects to date have been formed by a jointing tool or have been sawed. Depth has been normally a fourth of the depth of the shoulder slab. Most contraction joints placed to date have been sealed with a low-cost rubber asphalt compound.

Shoulder joints adjacent to jointed pavement have been made to match joints in mainline pavement, and intermediate joints have been added where necessary to stay within the short spacing criteria established.

Corrugated Rumble Strips

Corrugations have been built into most concrete shoulder projects to date. The rumble effect has proved to be a distinct safety feature (7), and corrugations have served to delineate lateral roadway limits. Corrugation widths of 4 and 6 ft have been used most consistently. Spacing between rumble strips has varied between 40 and 100 ft. One-inch deep corrugations have proved to be most effective in serving as a warning device to the driver. Individual corrugations have been rounded rather than peaked, and the tops of corrugations have been placed at or below pavement surface level to improve maintenance characteristics in areas where snow removal is a problem.

Rumble strips have been installed by hand on most projects. A new piece of equipment that forms those corrugations mechanically is being used on a project under construction in Illinois (Fig. 3). A similar machine will be used as an attachment to a full-width slip-form paver on a portion of that project in 1973.

Early experiments showed that rumble strips should be separated from transverse contraction joints. That keeps joints free of material that collects at the bottom of rumble-strip depressions and simplifies joint maintenance.

Although some embankment erosion has occurred adjacent to corrugated sections where the roadway is on a relatively steep grade, that problem does not appear to be significant. It has been provided for in one state by the addition of single corrugations at 20-ft intervals when longitudinal grades exceed 2 percent.

Reinforcement

With one exception, all concrete shoulders to date have been built without reinforcement. Two early experiments are adjacent to continuously reinforced concrete pavements on heavily traveled Interstate routes. There are only isolated instances of reflective cracking in the shoulder on those projects, and total shoulder cracking is insignificant. Performance to date does not indicate a need for steel reinforcement in concrete shoulders.

DESIGN REQUIREMENTS

Some minimum design requirements have emerged from experimental projects built to date and from the experience of the states involved.

Thickness

A 6-in. thickness is adequate. Where it is considered impractical to add and compact subgrade material to bring up the grade adjacent to pavement already in place, shoulder thickness can be made to match roadway pavement thickness at the longitudinal joint where they join. Likewise, where shoulder and roadway pavements are built integrally, shoulder thickness may be the same as main-line pavement at pavement edge and thinned to 6 in. at the outer shoulder edge.

Subbase

Concrete can be placed directly on in-place material; no special subbase is needed. Where the shoulder is placed integrally with roadway pavement, construction expediency may dictate the use of the same subbase as under the main-line pavement.

Tie Bars

Tie bars should be placed between main-line and shoulder concrete to keep the longitudinal joint tight and maintenance-free. Where the shoulder is placed alongside old pavement, tie bolts may be turned into expanding end anchors set in holes drilled in the edge of the old pavement slab. Tie steel should be designed for reduced shoulder width and thickness as discussed earlier.

Key

The key may be omitted without endangering the performance of the shoulder. However, improved structural capacity of both roadway and shoulder results from the use of a properly dimensioned key and tie-bar combination. Where feasible, use of a key is recommended.

Longitudinal Joint

The longitudinal joint between shoulder and roadway pavements where placed simultaneously may be formed by sawing or by inserting plastic of proper thickness and depth. Where the shoulder is placed adjacent to an in-place pavement, the vertical face of the pavement in place may be painted with bituminous material to aid in breaking the bond. Sealing of this joint is optional. The joint may be merely edged and no sealant reservoir provided; where sealing is specified, the reservoir need be no greater than $^3/_8$ in. wide x $^3/_4$ in. deep. A low-cost thermoplastic (rubber asphalt) may be used as a sealant.

Contraction Joints

Contraction joints in the shoulder should be extensions of the joints in main-line pavement if the 2 slabs are placed simultaneously. They should be of the same type and spacing. If the shoulders are placed separately, contraction joints may be grooved, sawed, or formed by plastic inserts. Shoulder joints should match those in the adjacent pavement, and intermediate joints should be added between them as required to maintain established spacing criteria—normally between 10 and 20 ft for plain concrete shoulders. Joint depth should be a fourth of the slab thickness. Randomization and skewing of shoulder joints are optional.

Concrete Mix and Reinforcement

Plain concrete without reinforcement may be used for shoulder pavement. Concrete mix can be the same as that specified for main-line pavement. If the roadway and shoulder pavements are built under the same contract, construction expediency may dictate that the shoulder design be the same as that of the main-line pavement, even though a design of less capacity might be structurally sufficient. If reinforcement is specified for both roadway and shoulder, the quantity of shoulder reinforcement can be reduced without endangering the performance of the shoulder.

Corrugated Rumble Strip

All concrete shoulders should include corrugated rumble strips impressed into the surface of the plastic concrete during construction. Corrugations should be 1 in. deep in a series 4 to 6 ft wide and spaced from 60 to 100 ft in rural areas. That spacing should be reduced where traffic is slower; perhaps a continuous rumble surface should be placed in certain ramp or interchange areas where driving on shoulders should be discouraged. In areas of the United States and Canada where snow removal is a normal maintenance operation, corrugations should be rounded rather than peaked and their highest elevation should be at pavement level or below.

Surface Texture

Surface texture of the concrete shoulder between corrugations should be such that maximum contrast is obtained with main-line roadway texture. Such contrast can also be obtained if color is added to the shoulder surface.

Shoulder Smoothness

Ridability is not a factor for shoulders. If planned roughness in the form of corrugations is introduced into the shoulder surface, surface tolerance for shoulder pavement can be relaxed from that required for roadway pavement. Surface deviations of $\frac{1}{4}$ to $\frac{3}{8}$ in. as measured from a 10-ft straightedge should be appropriate for shoulder pavement.

CONSTRUCTION

Equipment Innovations

Contractor-built paving rigs were used in early shoulder projects to consolidate and shape the concrete. On recent projects, innovations in equipment for placing shoulders have been introduced by the equipment and contracting industries. Small, self-propelled slip-form machines for placing 4- and 10-ft wide shoulders are now being used extensively (Figs. 4 and 5). Those machines are electronically controlled for grade and alignment and allow the rapid placing of shoulders adjacent to pavements in place. Full-sized slip-form pavers can be blocked out and converted to placing of narrower shoulder concrete. Shoulders can also be placed simultaneously with the roadway pavement. A specially built slip-form paving machine placed the 41 ft of pavement and shoulder width in a single pass on a portion of the East-West Tollway extension in western Illinois. Shoulder details for alternate methods of construction are shown in Figure 6.

The operation of placing tie steel has been simplified. Modern slip-form paving machines can be equipped with attachments to install tie bars so that they need not add greatly to shoulder cost or construction effort.

Where concrete shoulders are placed adjacent to old pavement, new developments in methods and equipment for anchoring new concrete to old are simplifying that procedure (6). Modern methods employ high-speed drilling equipment with multiple drills, shallower drill holes, and threaded metal anchors into which short hook bolts are turned.

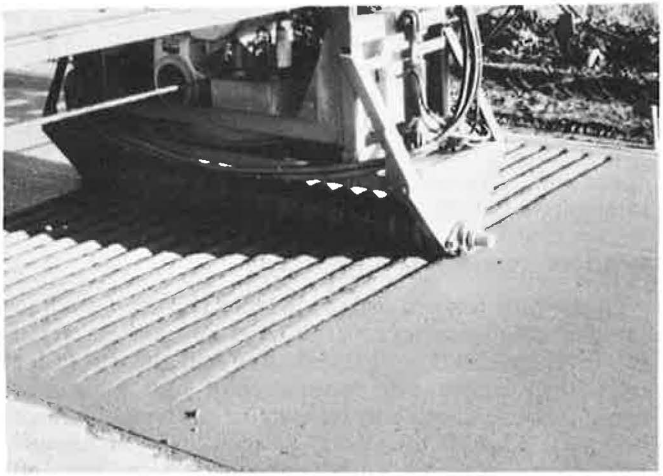


Figure 3. Machine placing corrugated rumble strip in concrete shoulder of East-West Tollway extension in Illinois.

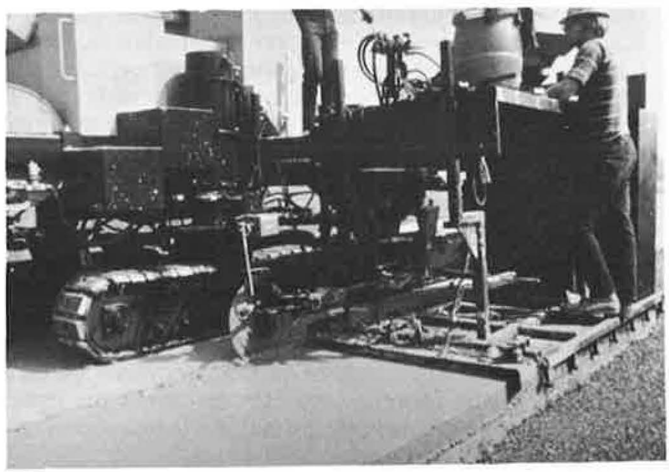


Figure 4. Slip-form paving machine placing 4-ft concrete shoulder on I-81 in Pennsylvania.

Figure 5. Slip-form paving machine placing 10-ft concrete shoulder on Barrington Road near Chicago.

That method replaces the old method of grouting in tie bars for half their length, a method that was time-consuming and less efficient.

The introduction of modern slip-form equipment and other innovations mentioned here has reduced the labor force necessary and the time required to complete shouldering operations. Shoulder construction, formerly a series of spreading, rolling, and surfacing operations in a restricted working area, is reduced to a single-pass operation. The hazard of injury to construction personnel is reduced, and the roadway is opened to traffic earlier.

Shoulder Texture and Delineation

A surface texture different from that in the adjacent main-line pavement is desirable for concrete shoulders. The HRB Committee on Shoulder Design reported in 1972 that the HRB Special Safety Committee had suggested that particular attention should be paid to "proper contrast or delineation of shoulders from the driving lane." If the shoulders are concrete, that contrast may be obtained by a difference in surface texture or color or by the addition of special delineation devices such as paint stripes, corrugations, or jiggle bars attached to the concrete or impressed into its surface while the concrete is plastic.

Texturing of concrete pavements has been accomplished with a variety of methods and materials—burlap drag, brooming, tine finish, combing, grooving, and special floats. All of those methods are applicable to concrete shoulders. For maximum contrast between shoulder and roadway, texturing of the shoulder by whatever method should be done in a direction different from that on the main-line pavement. Using a different method of texturing of the shoulder surface would also help to differentiate it from the traveled way. As an example, the magnesium float used in New York, which provides a serrated or lightly grooved surface texture, would furnish maximum contrast if applied transversely or on a skew on concrete shoulders adjacent to the main-line pavement that had been textured by broom or burlap drag in a longitudinal direction. Other finishing and texturing methods and combinations that would provide a contrast between the 2 areas need to be explored. Because of the relatively narrow width of the shoulders and their more liberal smoothness tolerances, hand-texturing methods can be used more extensively on shoulder surfaces and heavier or deeper textures can be applied than on the roadway. Because smoothness and ridability are not factors, the choices available for texturing concrete shoulder surfaces are limited only by the imagination of the builder or designer and by the need for ensuring surface durability of the concrete.

Color has been used on some projects to differentiate the traveled way from shoulders, acceleration and deceleration lanes, and ramp exits and entrances. A 2-course construction of the concrete shoulder was used on a 1972 shoulder project in North Carolina; the top course was a 2-in. thick red-colored concrete. Other states have used dry red coloring powder (iron oxide) worked into the top $\frac{1}{8}$ to $\frac{1}{4}$ in. of the concrete surface with long-handled floats. On a project on a primary highway in Nebraska recently, a bituminous liquid used as a cure for the concrete shoulder also furnished excellent contrast with the light-colored concrete pavement.

Several delineation methods and materials other than texture and color have been tried on shoulders. Corrugations impressed into the plastic concrete have been most popular. In Texas, several different experimental designs were tried on a shoulder project built in 1971. Seven different types of rumble strips were installed on the 7-mile project for evaluation. The Texas standard is a raised jiggle bar composed of 6-in. square ceramic sections attached to the shoulder surface for a 7-ft length on a 10-ft wide shoulder. In Alabama, an experimental shoulder project has been proposed in which delineation methods to be evaluated include 4 alternatives: full-width corrugations intermittently spaced, tapered continuous corrguations, raised pavement markers and paint stripes placed diagonally and intermittently spaced, and raised pavement markers continuously placed on edge paint stripe.

With normal shoulders, edge stripes must be placed on the edge of the roadway pavement. If concrete shoulders are used, striping can be done on the shoulder side of the longitudinal joint so that traffic can use the full width of the travel lane. Furthermore, if there are corrugated rumble strips in the concrete shoulder, the paint stripe would fall in the corrugations and result in improved visibility in inclement weather when flat paint stripes are largely invisible because of glare and reflection (8).

Experimental delineation methods such as those being tried in Texas and Alabama are examples of the type of research needed to arrive at the most positive and effective means to help keep vehicle drivers from straying from the roadway or mistaking the shoulder for an additional travel lane.

SHOULDER COSTS AND MAINTENANCE

Cost comparisons of various shoulder types are scarce because of the relatively small size and experimental nature of projects built to date and the short history of concrete use in that area. Until larger projects have been routinely awarded and built, true costs will be difficult to determine.

Maintenance information for concrete shoulders is largely limited to the experimental projects built in Illinois during the 1965-67 period. Other projects are so recent that they have not required maintenance.

That is apparently true of older projects as well. Duncan, reporting on the 1965 shoulder project on Star Route 116 (4) said, "After six years, the shoulders appear to be just as constructed with no signs of any maintenance problems to date." The performance of a later Illinois project was reported as follows (2): "The performance of the portland cement concrete (PCC) is significantly better than that of any of the other types. While service-life projections of perhaps 20 years based on less than 2 years of service cannot be made with a truly high degree of confidence, it would seem that, of the various types of paved shoulders included in the experiment, the PCC shoulders may have the best chance of serving the longest time without need for special maintenance."

SUGGESTED AREAS FOR FURTHER STUDY

Effects of Temperature and Moisture as Influenced by Color of Shoulder Surface

A paper published in 1969 (9) reported on variations in temperatures and moisture conditions in a pavement cross section occurring from use of shoulders with different colors. In that study, made in northern New York, 8-ft wide shoulders of 4-in. bituminous stabilized gravel were used adjacent to a 9-in. concrete pavement. Shoulders on one side were painted white. Temperature and moisture characteristics differed greatly at times between the areas influenced by the black and the white shoulder. The temperature of the area under the white shoulder was lower at all times than under the black. That differential became significant during a spring thaw period, during which the black shoulder became partially thawed while the white shoulder remained completely frozen. That condition had considerable influence on changes in the moisture cross section. A midwinter thaw of short duration substantially increased the moisture content under the black shoulder while the moisture content under the white side remained at mid-winter levels.

The study "clearly indicated that the black shoulder sustained a substantially higher average moisture content than the white shoulder, especially during the period from late January to early April" (9). The study concluded that "shoulder color influences greatly the temperature characteristics under road shoulders." The authors suggest that "the effect of a white shoulder remaining frozen longer than a black shoulder might be used to reduce the flow of meltwater from snowbanks to a position under the pavement. The observed accumulation of water at the edge of pavement (during a thaw period) is consistent with the often-observed phenomenon of pavement failure and weakening of material under the pavement edge."

Further study of concrete shoulders built as a part of recent experimental projects could establish whether the findings of the New York research are applicable to variations in moisture and temperature under concrete shoulders.

Figure 6. Typical pavement and shoulder sections of East-West Tollway extension in Illinois.

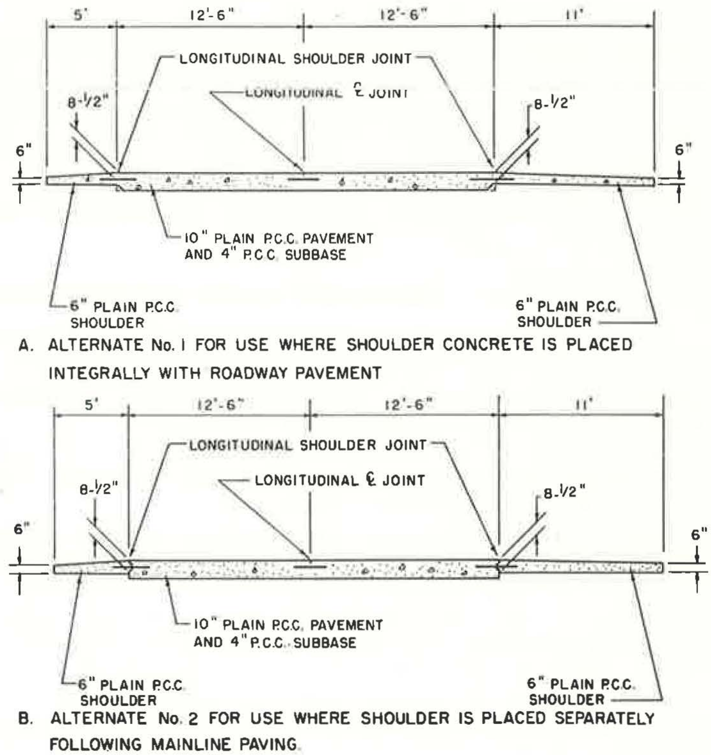
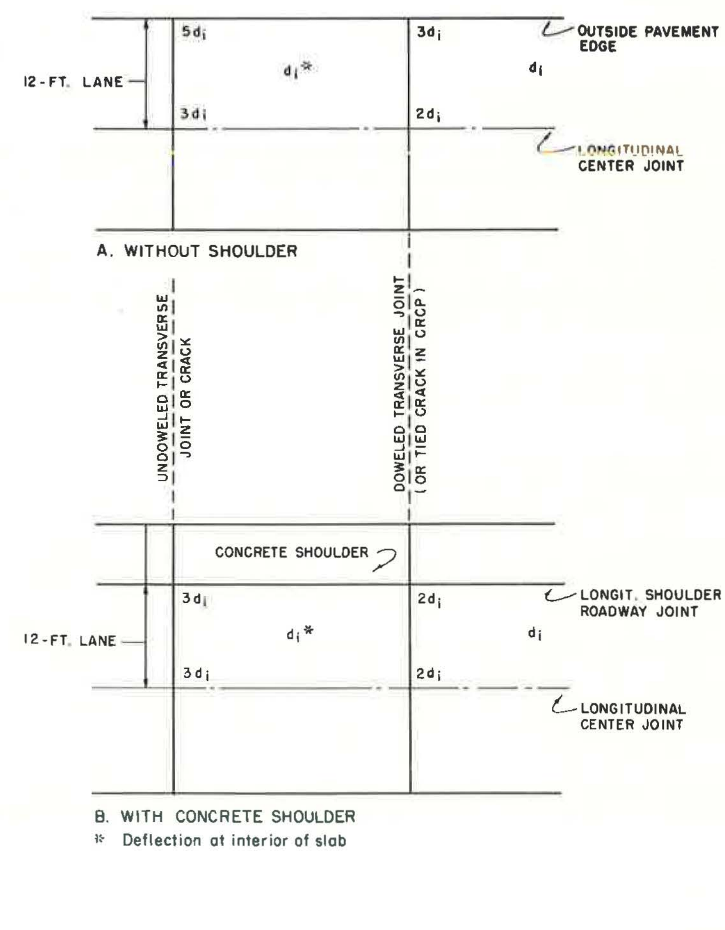


Figure 7. Deflections in concrete pavement as influenced by presence of tied concrete shoulder.



Effect on Pavement Structural Capacity of a Tied and Keyed Rigid Shoulder

Other conditions being equal, slab deflection at joints, cracks, and free edges is greater than the deflection at the interior of slabs.

Figure 7 shows the various load locations that occur on modern pavements under traffic. Relative deflections produced by a given load are shown for each location. The magnitude of those slab deflections is based on data from laboratory measurements (10). (It is assumed in these illustrations that load transfer across transverse joints or cracks is provided by aggregate interlock, by dowels, or by continuous reinforcement. Longitudinal joints are tied and keyed.)

Figure 7 shows the benefit of an attached rigid shoulder in improving the structural capacity of the pavement system by reducing deflection at the tied roadway-shoulder joint.

The data suggest that further full-scale deflection measurements should be made on projects where concrete shoulders have been built adjacent to concrete roadway pavement. Substantiating the research referenced here could lend support to a reduction in future pavement design thickness or to a greater expected service life for the present pavement system.

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