1126

INVESTIGATION OF PAVEMENT PROBLEMS ON I 275

TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION

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PROBLEMS ON I 275

Research Laboratory Section
Testing and Research Division
Research Project 79 F-158
Research Report No. R-1126

Lansing, October 1979

The I 275 freeway extends from I 75 in the south to its northern terminus at the I 696 interchange—a distance of approximately 39 miles. About nine miles south of I 696 it is joined by I 96 and this portion of the freeway is designated I 96/I 275. The general location and alignment of the freeway are shown in Figure 1, which also shows that the construction entailed 13 different construction projects. For the sake of simplicity, each project has been assigned a code number that will be used throughout this report whenever reference is made to a specific project. As can be noted, the code numbers range from 1 through 13 and are assigned consecutively from south to north.

The pavement is continuously reinforced concrete, 9 in. thick. The roadways on the I 96/I 275 portion are made up of four 12-ft lanes; whereas, on the remaining section they consist of three 12-ft lanes. The amount of steel reinforcement in the longitudinal direction is 0.7 percent on all projects. Transverse reinforcement was used on Projects 1, 2, and 9 through 13, but not on Projects 3 through 8. The pavement was placed on a foundation consisting of a 10-in. subbase layer of granular material overlaid with 4 in. of dense graded aggregate except on Project 1 where 14 in. of granular material was used instead of the two-layer combination used on the other projects. The pavement on Projects 1, 2, 10, 11, and 13 was placed in 1974 and on the remaining projects in 1976. The pavement from I 75 to Telegraph Rd (Project 1) was opened to traffic in November 1975, from Telegraph Rd to M 153 (Projects 2 through 9) in January 1977, and from M 153 to I 696 (Projects 10 through 13) in July 1975.

Longitudinal cracking was discovered during routine pavement surveys conducted in 1977. As a result, special surveys were made during May 1978 and again in March 1979. In addition, punch-out failures—small sections of concrete which are 'punched' into the base or broken apart by traffic—were found on three projects. Table 1 lists the percent of longitudinal cracking on each project for both the 1978 and 1979 surveys and the number of punch-out failures found in each survey.

The rather large increases in the amount of longitudinal cracking on some projects from 1978 to 1979 is believed to be due largely to the different survey procedure used. In 1978, the pavement was surveyed only from the traffic lane shoulder, and apparently cracks in the third or fourth lanes were not visible from that distance. In 1979, the survey was conducted from both shoulders and thus more accurate with respect to cracks in the inside lane or lanes. With respect to punch-out failures, another survey was conducted on May 31, 1979, with the results showing a total of 30 locations with this type of problem. Although the number of punch-out failures had increased since the March survey, they were still confined to Projects 4, 5, and 8.

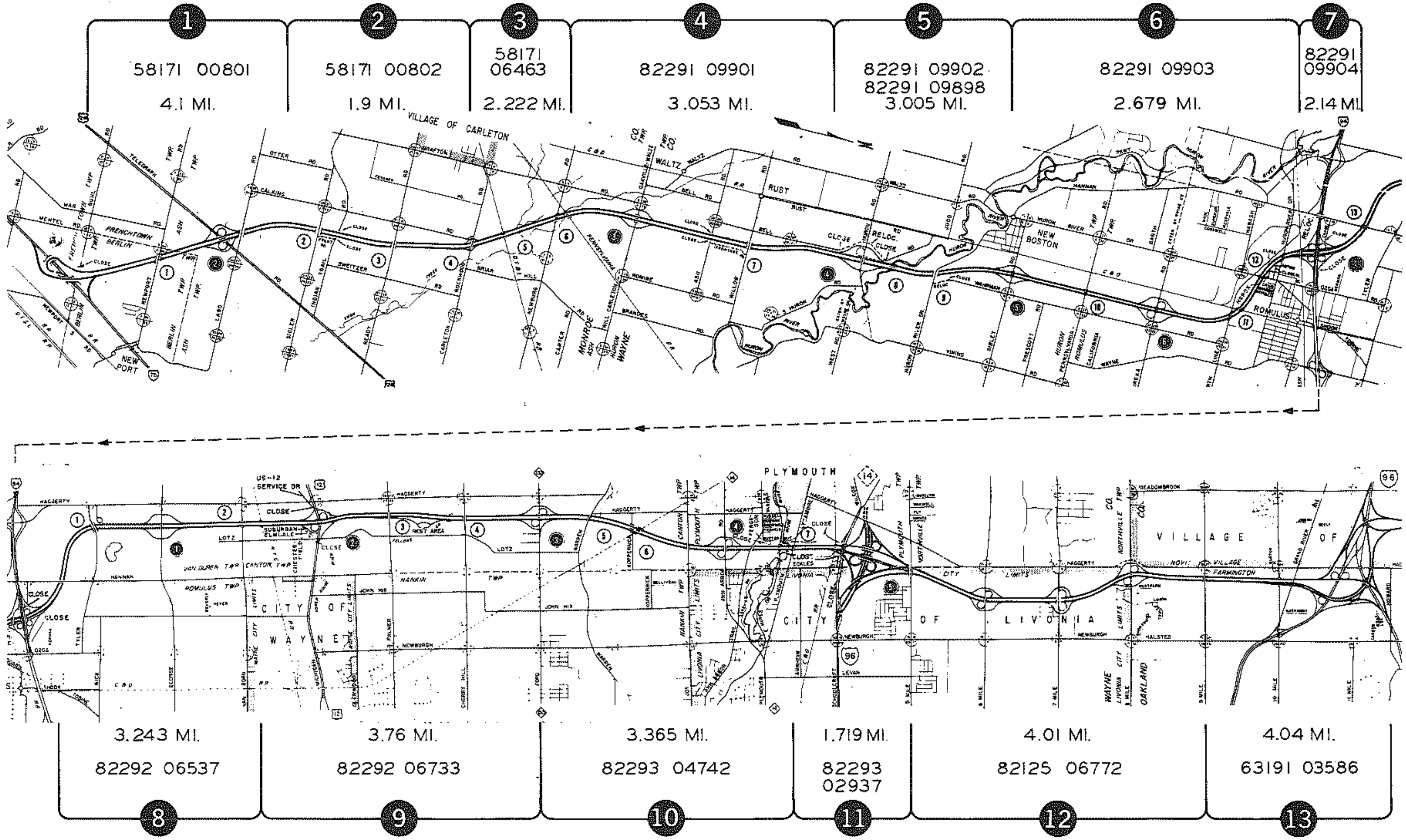

Figure 1. General location and alignment of I 275 and I 96/I 275 freeway. Mileage, contract numbers, and code numbers are also shown.

TABLE 1
SUMMARY INFORMATION ON I 275 PAVEMENT CONDITION

Project Code No.	Control Section	Job No.	Contractor		Approximate Project Location[1]	Paving Year	Equivalent Two-Lane Pavement, mi	Trans-verse Steel	Percent of Length Cracked Longitudinally[2]		No. of Punch-Out Failures	
			Paving	Grading					May 1978	March 1979	May 1978	March 1979
1	58171	00801	Denton	Holloway	I 75 to Labo Rd	1974	6.4	Yes	0.47	0.64	0	0
2	58171	00802	Edison	Edison	To Carlton-Rockwood Rd	1974	5.7	Yes	0.44	1.10	0	0
3	58171	06463	Eisenhour	Holloway	To Will Carlton Rd	1976	5.7	No	0.27	0.40	0	0
4	82291	09901	Eisenhour	Holloway	To West Rd	1976	9.1	No	7.71	14.80	3	6
5	82291	09902	Eisenhour	Holloway	To Sibley Rd	1976	8.5	No	2.66	12.83	2	5
6	82291	09903	Thompson-McCully	Edison	To C&O RR	1976	7.5	No	0.71	5.77	0	0
7	82291	09904	Ministrelli	Edison	To north of I 94	1976	6.1	No	1.66	7.62	0	0
8	82292	06537	Eisenhour	Edison	To Penn Central RR	1976	8.8	No	3.53	16.79	1	1
9	82292	06733	Eisenhour	[3]	To M 153	1976	9.8	Yes	--	1.25	0	0
10	82293	04742	Sargent	Holloway	To Plymouth Rd	1974	8.7	Yes	--	--	0	0
11	82293	02937	Denton	Holloway	To Five Mile Rd	1974	4.3	Yes	--	--	0	0
12	82125	06772	Eisenhour	B&V Construction	To Nine Mile Rd	1976	15.2	Yes	--	1.60	0	0
13	63191	03586	Denton	Holloway	To I 696	1974	8.9	Yes	1.16	3.61	0	0

[1] Project locations are from south to north.
[2] Percent based on length of equivalent two-lane pavement.
[3] Edison, Holloway, Kensington, and DiPonio.

Because of the unusually large amount of longitudinal cracking occurring on some projects, soil samples were taken from two locations in December 1978 and from five areas in March 1979 in an attempt to determine the causes of the cracking. Tests on these samples indicated that the soil foundation could be a factor in the causes leading to the formation of cracks. A more extensive investigation was proposed by the Testing and Research Division in May 1978 and was approved by the Department's Engineering Operations Committee at its June 6, 1979 meeting. This is a report of that investigation.

Scope

Since the nature of the cracking appears to be the same on all projects, and because of lower traffic volumes south of I 94, only the six most southern projects were selected for investigation. Because more cracking is occurring on the southbound roadway, and to further limit the test area, only this roadway was studied. Also, to ensure minimum interference with traffic, only the traffic lane (outside lane) was tested.

The selected test area, excluding bridges and standard pavement, contains 68,550 ft of three-lane pavement. This length of pavement was divided into 44 test sections, each 1,500 ft long and one sample site was selected 1,000 ft into each section. This constitutes a systematic sampling plan with a randomly selected starting point. In addition, eight sample sites were selected in cracked areas to obtain more data on distressed sections.

Each sample site was arbitrarily defined as representing an area three lanes wide and 200 ft long. The sample sites were divided into cracked and uncracked sites depending on whether or not there were any longitudinal cracks within the site area. On the basis of this division, there were 24 cracked and 28 uncracked sites.

The investigation deals with the performance of the concrete and the foundation, a review of the construction materials, and a discussion of crack locations. In addition, the lengths of cracks in selected test sections were measured so that future measurements can be made to determine whether the cracks are continuing to propagate. Moreover, a small-scale crack sealing study was made to determine the feasibility of sealing the longitudinal cracks.

Crack Locations

The crack locations with respect to project numbers and lanes, as of March 1979, are shown in Figure 2 for the six projects on the southbound roadway which were selected for this investigation. As can be seen in the figure, Projects 1 and 3 are free of cracks, but the remaining four projects all contain longitudinal cracking in varying amounts.

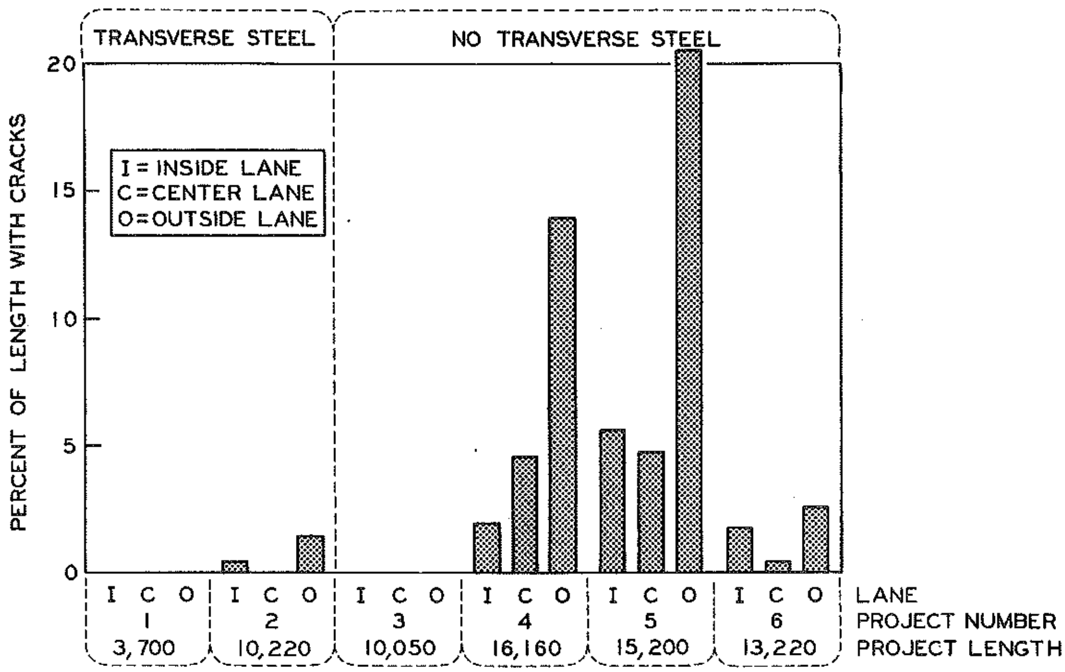

Figure 2. Percent of lane with longitudinal cracks on Projects 1 through 6 as of March 1979.

The lane with the most cracks is the outside (traffic) lane, ranging from 1.5 percent on Project 2 to 20.6 percent on Project 5. The inside lane on three projects is cracked more than the center lane; whereas, on Project 4 the reverse is true. Since most trucks travel the outside lane, traffic loads are suspected to be one of the factors contributing to the formation or propagation of longitudinal cracks.

Although the exact locations of cracks within a lane were not measured, most of them are located in the middle 8-ft portion of the 12-ft lanes. A number of cracks originate and end at the same lane edge, but the majority parallel the pavement edge. Some cracking zig-zags the sawed longitudinal lane joint, notably on Project 13 where 65 percent of the cracking in the southbound roadway is of this type. These zig-zag cracks are normally caused by late sawing, or loads on the slab before sawing the longitudinal joint. Where tie bars and transverse steel are used, they hold these cracks together; thus the cracks have only minimal effect on the pavement's performance.

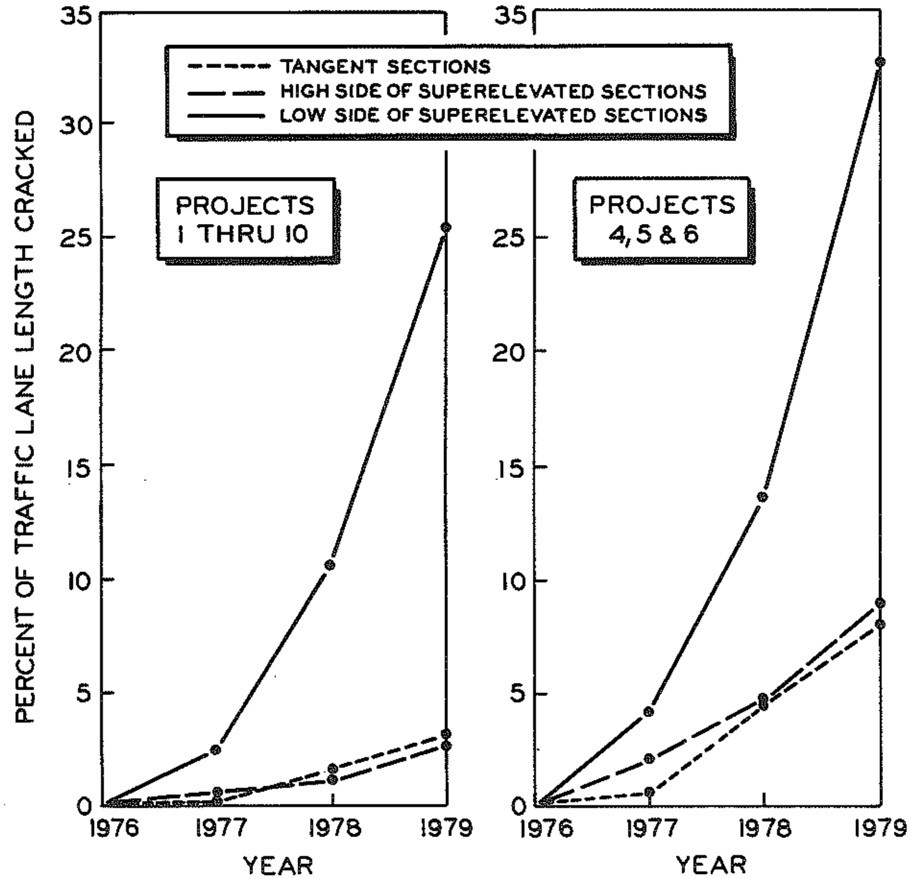

Figure 3. Percent longitudinal cracking in the traffic lane with respect to tangent and superelevated sections on three-lane pavements on Projects 4, 5, and 6 and on Projects 1 through 10, southbound roadway.

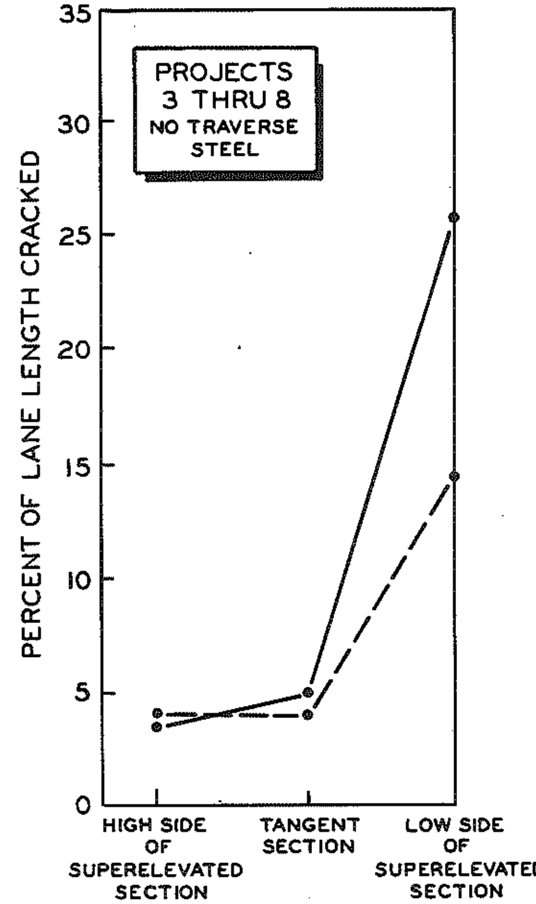

Figure 4. Percent longitudinal cracking on the traffic and inside lane on Projects 3 through 8, southbound roadway.

As can be noted in Figure 2, most cracking is located in projects without transverse steel, with the exception of Project 3 where no cracking has occurred. In terms of percentage, the pavement without transverse steel has cracked for a total of 2.7 percent of its equivalent lane length. The pavement with transverse reinforcement contains cracks totaling 0.3 percent of its equivalent lane length. Although the absence of transverse steel should not cause longitudinal cracking, its use will hold existing cracks tightly together, thus minimizing the progressive distress that is associated with cracked unreinforced concrete pavement.

The effect of the roadway alignment on cracking is shown in Figure 3. The graph at the left illustrates that for Projects 4, 5, and 6, the low sides of superelevated sections contain more than three times as much cracking as the tangent sections and the high sides of superelevated sections. The graph at the right shows that the same pattern is true for all the three-lane pavement on southbound I 275, except that there is about eight times as much cracking on the low sides of the superelevated sections as compared to the other alignment types. It should also be noted that the increase in amount of cracking has been much higher on the low sides of the superelevated sections than on tangent sections or on the high sides of superelevated sections. While this phenomenon is most dramatic for the traffic lane (Fig. 3), it is also true for the inside (median) lane (Fig. 4). The center lane, which cannot be classified as to high or low, nevertheless shows slightly more cracking on curves as opposed to tangents. These observations on the correlation of cracking with superelevation hold for both the percent longitudinal length cracked as well as for the number of new cracks occurring each year.

One difference between a tangent and superelevated section is the surface drainage, and in some cases the drainage path, in the base and subbase layers. In superelevated sections, the surface water runs across all lanes to the low side; whereas, on tangent sections the crown point is 24 ft from the traffic lane edge. On some curves the subgrade is designed to slope from one lane edge to the ditch bottom at the opposite side which increases the drainage length by 12 ft. It is suspected that the increased drainage path and the additional surface water available for entering the bases through the shoulder joint may be one of the reasons why more cracking occurs on the low side of superelevated sections.

Sampling

At each of the 52 sample sites, two 6-in. diameter cores were taken through the concrete slab at the center of the traffic lane. The longitudinal distance between the cores was dependent upon the transverse crack spacing, but normally ranged from 6 to 10 ft. The concrete cores were saved for laboratory tests.

Disturbed samples of the base and subbase and of the subgrade, to a maximum depth of 5 ft, were taken through one core hole and the thickness of the base and subbase layer was measured. Additional disturbed base material was collected through the second core hole prior to driving a 3-in. diameter by 36-in. long Shelby tube through the subbase and into the subgrade to obtain undisturbed samples of these materials. Shelby tube samples were also taken from the side slope when field conditions indicated they were needed to determine subbase drainability. Hand-auger borings were made on the slopes to determine subbase thickness and depth to the subgrade. The cross-sectional surface profile of the roadway from ditch line to ditch line was established at each sample site so that in conjunction with measured layer thicknesses, the 'as-built' cross-section dimensions would be available for drainability calculations.

Construction Materials

Records of materials used to construct the roadway section selected for investigation of longitudinal cracking were reviewed to determine if particular materials could possibly be identified as a cause of the distress.

The granular material used for the subbase came from several borrow pits located near the projects. On Project 1, 14 in. of granular material, placed in two 7-in. lifts, was authorized in lieu of separate subbase and base layers. Approximately half the length of Project 2 was built with subbase material from Willow Pit and the material for the other half was from a pit in Monroe County. The base for Project 2 was a limestone material from France Stone Pit.

The subbase for Project 3 was all from Willow Pit and the slag base was from E. C. Levy's Plant No. 3. For Project 4, the subbase came from a borrow pit on Willis Rd and the base from E. C. Levy's Plant No. 5. Two pits (Willow and Farrantino) were the sources of the subbase on Project 5, and the base source was E. C. Levy's Plant No. 3. The slag base material used in Projects 3, 4, and 5 was of the basic oxygen furnace type. Project 6 was built with subbase from the Willow and Zayty Pits and from Wayne County sanitary landfill, and the base consists of natural aggregate from Dixon Pit.

The cement used on all projects was Type 1A. Four brands of cement were used: Dundee on parts of Project 1 and all of Project 2; Peerless on

Projects 3 and 4; Medusa on Project 5; and Wyandotte on Project 6 and on parts of Project 1. Blast furnace slag coarse aggregate was used in the concrete for all six projects.

Except for a 2,600 sq yd area on Project 1, where a regular pavement grade concrete (six sack cement per cu yd mix and a 28-day strength of 3,500 psi in compression), a modified grade concrete was used on the entire southbound roadway portion of Projects 1 through 6. The modification consisted of using 5.6 sack cement per cu yd and adding a water reducer to the mix. The compressive strength design factor remained at 3,500 psi in 28 days. Tests on cores taken through the pavement indicate the slab thickness, steel depth, compressive strength, and seven-day modulus of rupture strength all met specification requirements.

On the basis of the material records review, there is no indication that a certain material is more likely to cause cracking than any other one. The effect on the performance of the concrete and the foundation, based on tests of samples from the existing roadway, will be discussed later in this report.

Pavement

The concrete cores removed from the pavement during the sampling work were tested in the laboratory to measure current strength levels. Of the two cores taken at each of the 52 sample sites, one was tested in compression and one in lateral shear at the level of the steel. Compression tests were in accordance with ASTM C42, where shear tests followed a Departmental procedure consisting of clamping the core on its side in a steel cradle and shearing off the top portion of the core through a similar steel cradle on the upper side of the core. The results of tests on the 104 cores are given in Table 2.

These data were analyzed and it was found that there was no significant correlation between cracking and either shear or compressive strength. There were only two compression tests out of 52 which were below 3,500 psi and this consisted of one each from Projects 4 and 6.

The average compressive strengths of cores from uncracked areas of Projects 4, 5, and 6 were 5,190, 4,610, and 4,730 psi, respectively, as compared to averages of 4,640, 4,800, and 4,670 psi for cracked areas. Similar shear strength averages for uncracked areas of Projects 4, 5, and 6 were 600, 610, and 775 psi, respectively, as compared to averages of 665, 660, and 685 psi for cracked areas.

The results of these tests on the 104 cores shown in Table 2 indicate an acceptable level of compressive and shear strength for all six projects sampled. Also, there does not appear to be an inherent weakness in cracked areas as compared to noncracked areas. The average pavement thickness and depth of steel are acceptable for all six projects cored, with the exception of the steel as measured on the four cores of Project 1 which were less than optimum.

TABLE 2
SUMMARY OF CONCRETE CORE TESTS

Project No.	Pavement Thickness, in.			Steel Depth, in.			Shear Strength, psi				Compressive Strength, psi			
	High	Low	Avg	High	Low	Avg	No. of Cores	High	Low	Avg	No. of Cores	High	Low	Avg
6	10.1	8.8	9.5	4.5	3.0	3.7	9	925	530	745	9	5,720	3,200	4,710
5	9.5	8.6	9.2	4.9	3.0	3.9	14	825	460	650	14	5,800	4,110	4,760
4	9.5	8.8	9.1	4.8	2.5	3.9	12	760	520	635	12	5,700	3,460	4,870
3	9.5	9.0	9.3	4.3	3.5	3.9	6	895	445	675	6	5,670	4,790	5,110
2	9.5	8.9	9.2	4.2	3.1	3.6	9	910	700	773	9	6,410	5,170	5,670
1	9.5	9.0	9.2	2.5	1.8	2.0	2	790	770	780	2	5,340	4,930	5,140

Foundation

The laboratory tests conducted on the foundation samples included determination of gradation, Atterberg Limits, AASHTO classification, specific gravity, layer thickness, density, moisture content, permeability, frost susceptibility, and drainability for those layers where each test was pertinent.

Because of time restrictions and limited test equipment available, it was decided early in the test program to complete tests on samples from a project with a large amount of cracking and from one with little cracking, in order to possibly establish the cause of the difference in their performance. Projects 3 and 5, with 0 and 20.5 percent cracking in the outside lane, respectively, were selected for initial testing. Although tests on samples from the other projects are still in progress, it is anticipated that conclusions reached, on the basis of the data from Projects 3 and 5, will not be changed. Test data are summarized in Tables 3, 4, and 5 for the subgrade, subbase, and aggregate base, respectively.

For 19 of the 20 sites, enough test data were available for statistical examination in the form of stepwise multiple regression of some test and environmental variables. The following variables were examined for their linear impact on longitudinal cracking accumulated by 1979 within each 200-ft site:

1) Tangent-nontangent sections,
2) High-low portion of superelevation sections,
3) Percent saturation of subbase,
4) Permeability of subbase,
5) Time between subgrade completion and concrete pour,
6) Time between base completion and concrete pour.

Low portion of superelevation was the only significant (0.10) variable selected by the stepwise procedure. The raw correlation of this variable with cracking was 0.71. None of the other variables taken singly or in combination could improve this enough to suggest their consideration as causal factors. Although the number of samples was low (19), it is not expected that additional samples will substantially alter these findings.

As mentioned earlier in the report, it was concluded that, on the basis of core strengths, the concrete was not a factor in the formation of longitudinal cracks. Although the statistical analysis failed to reveal any correlation between the foundation variables examined and cracking, it is evident the mechanisms causing cracking must lie in the pavement support layers. On the basis of examination of the test data, coupled with observations in the field during sampling, the following discussion will attempt to establish the role of each support layer in the sequence of events leading to the formation of cracks.

<u>Subgrade</u> - A summary of the subgrade test data is given in Table 3. As the data show, the subgrade on Project 3 is classified as A-6; whereas, on Project 5 it consists of three classifications: A-2-4, A-4, and A-6. The A-2-4 soil is basically a fine sand and generally provides a stable subgrade. The A-4 and A-6 soils are predominantly clayey silt materials and both are commonly used for fills. They both perform well when moisture contents are low.

During sampling it was noted that the clayey subgrade material was very firm, dense, and relatively dry. This was also the case at two borings made to a depth of 8 ft during an earlier cursory study. As can be seen in the table, the in-situ moisture contents were below the plastic limit of the soil which indicate a very stable subgrade material, behaving essentially as an elastic material. Being well compacted and in an elastic condition to a considerable depth, the subgrade should not be a factor in the formation of longitudinal cracks.

TABLE 3
SUMMARY OF SUBGRADE DATA

	Station No.	Sample Site No.	Liquid Limit	Plasticity Index	Plastic Limit	Moisture Content, percent	-200 Material, percent	AASHTO Classification
Project No. 5	848+00	10	--	N.P.	--	--	38.0	A-4
	833+00	11	--	N.P.	--	--	19.3	A-2-4
	818+00	12	--	N.P.	--	--	17.0	A-2-4
	810+00	12a	24	10	14	11.7	68.0	A-4
	803+00	13	24	10	14	10.0	72.0	A-4
	787+00	14	27	11	16	13.1	68.0	A-6
	783+00	14a	26	11	15	12.1	68.0	A-6
	778+00	14b	26	11	15	12.6	65.0	A-6
	773+00	15	27	12	15	12.2	71.0	A-6
	762+00	15a	25	11	14	11.8	67.0	A-6
	758+00	16	26	12	14	11.1	69.0	A-6
	736+00	17	24	10	14	9.2	64.0	A-4
	721+00	18	30	13	17	14.9	84.0	A-6
	704+00	19	26	12	14	12.9	78.0	A-6
Project No. 3	526+00	31	27	13	14	10.4	70.0	A-6
	501+00	32	30	14	16	12.1	72.0	A-6
	486+00	33	40	20	20	16.8	78.0	A-6
	467+00	34	29	13	16	11.7	67.0	A-6
	452+00	35	35	16	19	16.2	87.0	A-6
	439+00	36	39	19	20	17.4	88.0	A-6

Subbase - One of the purposes of the subbase layer is to provide adequate drainage of water entering the foundation through joints and cracks in the surface. While many factors influence the amount of water a subbase layer can drain, it is obvious that the more open-graded the material, the better drainage it provides. Further, the exposed surface of the subbase on the side slopes must be kept open to prevent blockage of the drainage outlet.

The test data on the subbase have been summarized in Table 4. As can be noted, the amount of -200 material contained in the subbase samples varied considerably—ranging from 2.6 percent at Site 18 to 22.1 at Site 32. Generally, the more fines a porous material contains the more dense it will be, which reduces its ability to drain. This may be noted by comparing the -200 material content with the permeability values given in the table. In general, the higher the fines the lower the permeability values. Although the permeability values are low compared to values ranging from 800 to 1,000 ft per day, which are considered essential for effective drainage, they are within the range of 1 to 80 ft per day which is normal for many subbase materials commonly used.

TABLE 4
SUMMARY OF SUBBASE DATA

Project	Sample Site No.	Super-elevation, ft/ft	Length of Longitudinal Cracking, ft	Thick-ness, in.	-200 Material, percent	Frost Susceptibility, mm/day	Saturation, percent at 2 to 3 in. tension	Perme-ability, ft/day	Effective Porosity, N_e	Dry Density, lb/cu ft	Drainage Restricted	
											Outside	Median Side
Project No. 5	10	0	0	13.5	7.5	0.20	88.5	17.4	0.04	108.00	No	No
	11	0	0	13.0	5.0	0.20	92.8	15.1	0.02	109.25	No	No
	12	0.02	0	10.8	3.5	0.00	93.9	19.2	0.02	109.88	Yes	Yes
	12a	0.02	125	13.5	10.8	0.04	93.8	3.4	0.03	111.75	Yes	No
	13	0.02	25	11.5	14.7	No Data	89.1	5.0	0.03	114.25	Yes	No
	14	0	82	11.0	12.4	0.53	No Data	No Data	No Data	111.75	Yes	Yes
	14a	0.02	165	10.0	13.6	0.00	100.0	9.0	0.00	114.87	Yes	Yes
	14b	0.02	143	9.0	6.9	0.40	81.1	31.9	0.06	111.75	Yes	No
	15	0.02	125	11.0	11.1	0.13	96.9	6.1	0.01	111.75	No	Yes
	15a	0.02	160	11.5	5.9	0.26	89.6	39.1	0.04	108.00	Yes	Yes
	16	0.02	50	12.5	6.4	0.00	85.3	8.2	0.05	110.50	Yes	Yes
	17	0	65	10.2	3.3	0.00	91.9	19.8	0.03	107.38	Yes	Yes
	18	0	127	10.2	2.6	0.00	95.4	14.1	0.01	111.13	No	Yes
	19	0	38	11.8	4.1	0.20	83.7	14.7	0.06	113.00	Yes	Yes
Project No. 3	31	0	0	12.0	8.0	0.00	88.0	13.7	0.04	108.63	Yes	No
	32	0	0	10.0	22.1	0.40	93.5	3.6	0.02	113.62	Yes	Yes
	33	0	0	12.5	11.1	0.13	96.9	3.3	0.02	113.00	No	No
	34	0	0	14.0	8.6	0.00	95.2	7.0	0.01	112.37	No	Yes
	35	0	0	11.0	7.8	0.00	92.7	12.3	0.02	114.25	Yes	Yes
	36	0	0	9.5	8.7	0.13	92.4	8.0	0.02	110.50	Yes	No

The subbase at 12 of the 20 sites was found to be over 90 percent saturated with the remaining sites having moisture contents in the 80 percent range. This is not unusual for sands, but it indicates that they have high capillary potential and do not gravity drain very effectively.

Another drainage problem discovered during soil sampling was that from the edge of the shoulder to the toe of the subbase, drainage is usually restricted because layers of poor draining materials have been incorporated in the subbase beyond the shoulders, and the side slopes have been covered with topsoil and clay. In such cases, the subbase cannot provide positive drainage, tending instead to trap water under the pavement. On the basis of field observations and permeability tests on subbase material from the slopes, it was determined (see table) that only three sites had unrestricted drainage, eight sites were restricted on one side, and nine were restricted on both sides.

The frost susceptibility test values range from 0 to 0.53 mm per day. These data show that the subbase is not frost susceptible in the usual sense, but it cannot remove water fast enough to avoid the intrusion and holding of surface water which, upon freezing, could cause heaving equal to about 10 percent of its thickness, due to the expansion of water during freezing.

It is evident that the subbase is not performing as intended. Although the permeability values are in the range found for many sands, pore sizes are such that it cannot gravity drain very well and the drainage outlet at the side slopes is restricted due to contamination and construction methods. It is believed that the availability of water in the subbase is part of the reason longitudinal cracking has developed. First, it provides water for formation of ice lenses in the base during freezing. Further, its moisture content may vary in accordance with variations in pore sizes which could cause differential heave when the subbase freezes. Since more water must be drained at the low side of the superelevated sections, it is plausible that in these areas the moisture problem is most severe and causes the more severe cracking at sample sites on the low side of the superelevated section (see table). Although not shown by the test data, it is generally accepted that loss of support occurs during, and for a period of time after, thawing which certainly would aggravate the crack problem.

Base - The base is a drainage layer and is intended for rapidly draining surface water away from the slab and into the subbase. Thus, it essentially is a vertical drain; whereas, the sand is a horizontal drain. As can be noted in Table 5, which is a summary of base test data, all samples tested except one, were found to be impervious, that is, the permeability is less than 1 ft per day.

TABLE 5
SUMMARY OF BASE DATA

Project	Sample Site No.	Thickness, in.	-200 Material, percent	Frost Susceptibility, mm/day	Permeability, ft/day
Project No. 5	10	4.00	5.6	2.38	Not Tested
	11	3.75	5.5	2.15	Not Tested
	12	4.00	5.6	1.92	Impervious
	12a	4.50	6.5	3.06	Impervious
	13	4.25	5.0	1.07	Not Tested
	14	4.25	4.6	1.59	Impervious
	14a	4.25	6.4	1.99	Impervious
	14b	4.00	6.2	1.59	Impervious
	15	3.75	5.2	1.19	Impervious
	15a	3.75	7.9	2.03	Impervious
	16	3.50	5.7	1.06	Erroneous Test Result
	17	4.50	2.6	2.04	1.8
	18	4.00	5.4	2.25	Impervious
	19	6.25	3.7	1.82	Impervious
Project No. 3	31	4.00	8.8	3.28	Impervious
	32	3.25	10.2	3.40	Impervious
	33	3.50	9.3	4.07	Impervious
	34	3.75	9.0	2.18	Impervious
	35	3.75	8.2	2.70	Impervious
	36	4.00	4.7	2.54	Impervious

Frost susceptibility data (Table 5) show a range from 1.06 to 4.07 mm per day. Frost heave values from 1 to 2 and from 2 to 4 mm per day are classified as low and moderate, respectively. On that basis, 12 sites had moderate and 8 had low frost heave potential. It should be noted that all samples from Project 3 were in the moderate group. Frost heave measurements taken during the earlier studies indicated that the pavement is subject to differential frost heaving, especially at the pavement-shoulder joint.

Although field observations indicated the base to be dense, the -200 material contained in the test samples was in the normal range for base materials. Its pore size is typical to that of silt which gives it high capillary potential.

The base conditions found during this study indicate that the base is a factor in the causes of longitudinal cracking. It certainly is a very poor drainage layer and it is frost susceptible. Water drawn into the base during freezing causes ice lenses to form. Since it is very likely there is a difference in the quantity of water available to cause frost heave, differential frost heave will result. Differential frost heave along the pavement-shoulder joint may occur as a result of additional moisture in the form of saltwater available during de-icing periods, which may account for the cracking in the outside lanes. The large amount of cracking found on the low side of curves may also result from differential heave since more saltwater and water, in general, is available at these locations. Unequal support conditions during thawing is thought to cause cracking in the center lane and could also account for some cracking in the outside lanes. During thawing, the base is in an ideal condition for pumping fines which may be the cause of faulting beginning to develop at some of the wider cracks. Pumping may also be a factor in the formation of punch-outs.

Crack Propagation

In order to determine if existing cracks continue to increase in length, cracks in 16 of the 44 test sections were selected and measured while the lane was closed for sampling work. Since future measurements will be done under traffic along the shoulder edge, cracks in test sections located in ramp areas were not selected because of the hazard involved in making future measurements in these areas. Only cracks in the traffic (outside) lane were included. Cracks terminating at the shoulder or at the interior lane joint were not considered for measurements.

A total of 26 cracks were measured—two each in 10 test sections and one in each of the remaining test sections. With respect to the projects, four cracks are located in Project 6, twelve in Project 5, nine in Project 4, and one in Project 2. The cracks are identified by reference to the stencilled station numbers along the pavement edge. To ensure that the selected cracks have space into which they can propagate, the length of uncracked slab at each crack end was measured except where the distance to the next crack was more than 100 ft in which case this fact was merely noted. Although it was proposed to measure the crack lengths quarterly, it is suggested that measurements be made only in the fall and spring of each year.

Crack Sealing

Since excessive water in the base and subbase is detrimental to pavement performance, sealing the longitudinal cracks to minimize the intrusion of surface water would be advisable. Therefore, a study is being conducted to monitor the horizontal and vertical movement of several cracks to develop sealing procedures and to evaluate several sealants.

A 1,500-ft section of southbound I 275 in Project 4 was selected for the study. This section has 12 cracks in the traffic lane with a combined length of about 1,000 ft. The test area is located between Sta. 623+00 and 637+50. The cracks vary from hairline to 1/4 in. in width with some of the wider ones beginning to develop faulting and spalling.

One to three sets of measuring plugs were set at each crack to monitor any movement. Initial measurements were taken to determine the horizontal distance between the two plugs in each set and to determine the vertical relationship between the two plugs. These measurements will be repeated this fall, winter, and spring to determine the movement at each crack in the test section. The length of each crack was also measured to monitor any increase in length that may occur.

An inspection of the cracks and a review of the available sealing materials indicated that a groove or reservoir would have to be formed along the crack to receive any of the sealants. Two methods were used to form the groove. The first method was close-range sandblasting with a 3/16-in. diameter nozzle. This method works quite well on the narrower cracks in conjunction with a full-depth sealant where the groove acts only as a reservoir to channel the sealant into the crack. However, it was found to be difficult to maintain a uniform depth. This problem might be reduced by mounting the sandblast nozzle on a wheeled cart. Another problem is, of course, removal of the sand from the roadway.

The second method was the use of a commercially available crack cutter or router. This was a MacDonald 'Crack Chaser,' Model GC1, made by the MacDonald Air Tool Corp. of South Hackensack, New Jersey. It is simply a wheel-mounted pneumatic hammer equipped with a face and side cutting bit. After some experimentation with the cutting capability of the router it was decided that a 1/2-in. wide by 1/2 to 3/4-in. deep groove would be the most practical size to use. This size groove was cut at a rate of about 120 ft per hour. The groove was quite uniform in width and depth and would be satisfactory for surface sealants but not for full-depth sealants because the crushed concrete was forced into the narrower cracks and could not be removed by either sandblasting or compressed air.

Since the amount of movement at the cracks is unknown, several sealants of two general types (shallow and full-depth) were selected for test installations. Light gray polyurethane, polysulfide, and silicone were used as shallow or surface sealants intended to seal only the top 1/2 to 3/4 in. of the crack. Flexible polyesters were used as full-depth sealants. A summary of the sealants installed is given in Table 6.

It is anticipated that after exposure to traffic and weathering for one winter, recommendations concerning sealing of the cracks can be made.

TABLE 6
SUMMARY OF SEALANTS USED

	Sealant Type	Brand	Groove Treatment	Amount Sealed, ft	Remarks
Shallow Sealants	Polyurethane, two-component, self leveling	W. R. Meadows Pourthane	Formed with router, primer applied, crack in bottom sealed with non-sag polysulfide.	100	For optimum performance where movement is significant, a plastic foam bond breaker would be required which would require a deeper groove.
	Polysulfide, two-component, self leveling	Sika Chemical Corp. Sikaflex 411	Formed with router, primer applied, crack in bottom sealed with non-sag polysulfide.	45	Same remarks as for polyurethane apply.
	Polysulfide, one-component, non-sag	Products Research Corp. PRC 7000 W. R. Meadows CM 60	Formed with router, primer applied, foam rod bond breaker used where the groove was deep enough.	95	Same remarks as for polyurethane apply. Material must be tooled thus requiring repair of spalls.
	Silicone, single-component, non-sag	Dow Corning Corp. Dow Corning 888	Formed with router, no primer required, no bond breaker was used.	40	Same remarks as above.
Full Depth Sealants	Polyester, medium viscosity	Preco Gold Label Flex	Formed with router, plastic foam was forced into crack to conserve material.	345	A low viscosity polyester is mixed with a dry activator and filler to adjust to a higher viscosity. Requires hand pouring and multiple applications to fill.
	Polyester, low viscosity	Reichhold Chemicals, Inc. Polylite 31-851	Formed by sandblasting.	40	A low viscosity polyester is mixed with a liquid activator. Several hand pourings were required to fill because its low viscosity permitted deep penetration, probably even into the base.
	Polyester, low viscosity, portland cement added	Reichhold Chemicals, Inc. Polylite 31-851	Formed with router.	120	Portland cement was added to the above material to increase the viscosity. The cement was difficult to incorporate without lumps of unmixed cement.
	Polyester, low viscosity, sand added	Reichhold Chemicals, Inc. Polylite 31-851	Formed with router.	120	Sand was mixed into the above material with more ease than the portland cement.

Conclusions

On the basis of field observations and test data obtained during this investigation, it is concluded that the concrete and subgrade are not factors contributing to the formation of longitudinal cracks. Although the exact mechanism responsible for cracking was not determined, it is concluded that the primary causes are attributed to poorly drained bases and subbases, blocked drainage paths, and frost susceptible bases. This condition results in excessive pressure on the slab due to freezing or a subsequent reduction in base support, or both, which induces large stresses in the slab when subjected to traffic loads. Since the steel reinforcement in a continuously reinforced pavement is constantly under a high tensile stress, there is considerably more tendency for the pavement to span reduced support areas in the base and act as a 'bridge' rather than a pavement with uniform support. As a consequence, the performance of continuously reinforced pavement is extremely sensitive to foundation characteristics.

The investigation failed to establish any reason why Project 3 is without cracks. However, it should be noted that relatively long crack-free sections exist on other projects also. Apparently, the many variables required to cause cracking are present in the right combination at only certain localized areas. Once a crack has formed and additional moisture enters the bases, the conditions change and the crack may propagate or a new one form in the near vicinity.

Recommendations

1. Drains should be placed under the pavement-shoulder joint in order to intercept surface water and to collect lateral draining subbase water. Tentatively, it is recommended that drains be placed on Projects 4 and 5 and on the northbound roadway of Project 8. It is suggested that details concerning drain design and placement be determined in cooperation with the Design Division.

2. Subject to the evaluation results of the cursory sealing study, it is suggested that the longitudinal cracks be sealed.

3. Punch-out areas should continue to be maintained using bituminous material until drain and crack sealing work is scheduled. At such time, concrete repairs should be made at severe punch-out locations.

4. With respect to future construction, it is suggested that the use of more open-graded materials in the base and subbase be considered. Also,

the use of edge drains should be considered to more effectively drain the bases. In addition, action should be taken to ensure unblocked subbase drainage outlets.

Additional Study

It is suggested that a study be initiated to determine if voids occur at the longitudinal cracks and at the pavement-shoulder joint, and to explore the possibility of undersealing to prevent or minimize the void formation should it be found that a problem exists.

Also, elevation measurements would be taken at scheduled intervals and at selected locations to determine slab movement due to frost action.