1152

BASE AND SUBBASE PROPERTIES AFFECTING
LONGITUDINAL CRACKING OF I 275

MICHIGAN
DEPARTMENT OF TRANSPORTATION

TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION

BASE AND SUBBASE PROPERTIES AFFECTING LONGITUDINAL CRACKING OF I 275

Research Laboratory Section
Testing and Research Division
Research Project 79 TI-622
Research Report No. R-1152

Lansing, April 1981

INTRODUCTION

Between 1975 and 1977, approximately 39 miles of I 275, a continuously reinforced concrete pavement, were opened to traffic. Longitudinal cracking of the pavement was discovered during routine pavement condition surveys conducted in 1977. By 1979, surveys indicated that some test sections contained longitudinal cracks equal to 7 percent of the equivalent lane mileage and some punch-out failures had occurred. The pavement foundation was the suspected cause of the cracking, so two preliminary foundation investigations were conducted, one of which has been reported (1). Although these studies indicated that several foundation problems did exist, no definite conclusion as to the cause of cracking was reached.

An intensive investigation of the test sections (Fig. 1) was conducted in 1979 in an effort to determine the specific cause of the longitudinal cracking (2). Because of time restraints, however, foundation analysis for only two of the six test sections investigated could be included in the report. From these limited data it was not possible to reach specific conclusions as to the cause of the longitudinal cracking.

The purpose of this report is to present the completed foundation study results, to provide a more in-depth analysis of the drainability and frost susceptibility characteristics of the base and subbase layers, to provide a more definite explanation of the cause of longitudinal cracking, and to suggest how the occurrence of additional longitudinal cracking may be attenuated. The additional foundation data and conclusions of this report do not alter the conclusions drawn from the earlier investigation (2), but they present additional information that could affect the manner in which future corrective action should be undertaken and be of value to the construction of new CRC pavements.

In the past, Michigan has not experienced a longitudinal cracking problem in CRC pavements. Therefore, transverse steel has frequently been omitted, in order to reduce construction cost, because its only purpose is to prevent longitudinal cracks from opening up should they occur. Two of the test sections included in this study (Sections 1 and 2) contain transverse steel while Sections 3 through 6 do not. The occurrence of longitudinal cracks in CRC pavements having no transverse steel and which have deficient foundations, results in serious differential pavement surface movement in which the cracks open and fault and punch-outs occur where longitudinal cracks intersect. This report deals primarily with foundation conditions which are thought to be responsible for the development of longitudinal cracking.

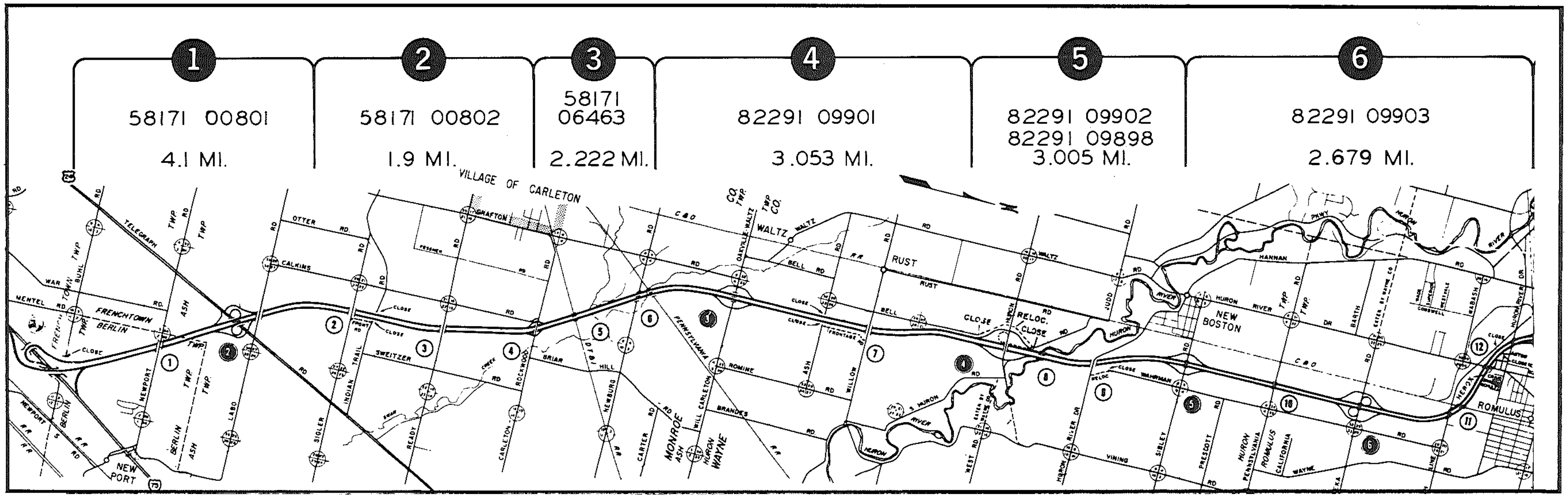


Figure 1. Section of I 275 investigated showing test section numbers, contract numbers, and mileage.

Figure 2. Clay and topsoil cover which restricts drainage at toe of the subbase.

FIELD INVESTIGATION

Sample sites were located at 1,500-ft centers and at intermediate areas wherever significant longitudinal cracking occurred. The as-built cross-section of the pavement was, where possible, established at each site and samples of base, subbase, and subgrade obtained from under the center of the traffic lane and at the side slopes. Level readings were taken to establish the ditchline-to-ditchline profile of each test site.

For a majority of sites in the six test sections included in this study, the side slopes are covered with clay and/or topsoil which restricts drainage of the subbase layer. In many cases a clay cover of 6-in. or more was found (Fig. 2). In Section 1, removing the clay side slopes allowed free water to drain out of the 14-in. coarse limestone base, as shown in Figure 3. In addition, the side slope area usually contains waste dense-graded base material, topsoil, clay and other poorly drained materials, so that subbase drainage is also restricted internally by the presence of these relatively impervious materials. The subbase layer under the pavement was, generally, a uniform sand. Surface infiltrating water tends to be trapped under the pavement due to its inability to flow laterally through the impervious side slopes. This problem is further aggravated by the flat side slope which further retards subbase drainage. The trapping of water under the pavement was observed in several cases where, after a rain, the free water level was at the bottom of the CRC slab (Fig. 4).

It was also found that in all areas where the subbase was properly constructed, with sod covering the exposed face of the subbase layer, vegetative cover was sparse to completely barren from the toe of the subbase layer to the ditchline (Fig. 5). Analysis of soil from sparsely vegetated areas indicated salt contents as high as 6 percent. The vegetative die-off could not be caused by surface run-off of deicing salts since there is good vegetative cover from the toe of the side slope up to the outer edge of the shoulder. The large salt concentration and vegetative die-off occurred where the subbase has poor drainability and/or restricted internal drainage. The general sequence of events is that saltwater infiltrates the pavement surface during the winter and accumulates in the subbase layer. Later, during the growing season, the saltwater slowly seeps out the toe of the subbase layer where evaporation causes such a high salt concentration that it kills the vegetative cover.

It was also noted that where the entire side slope area is covered with clay and topsoil there are no areas barren of vegetative cover. In this case, however, the vegetative cover tends to be thin and, under close inspection, appears to be growing under stressful conditions. In such cases,

Figure 3. Wet condition of the 14-in. limestone base is indicated by removal of clay side slope.

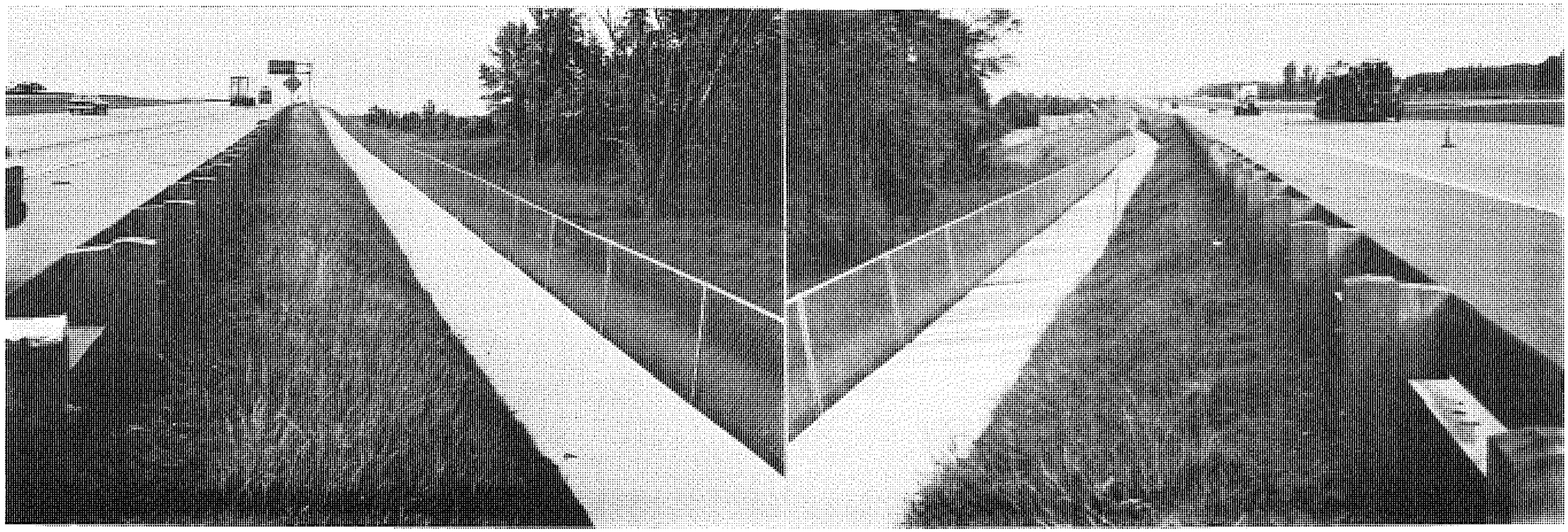
Figure 4. Free water, seen at the bottom of the core hole, is located about 9 in. from the pavement surface a day after a summer shower.

Completely barren side slope and subbase erosion.

Sparse vegetative cover and no subbase erosion.

No vegetative cover or subbase erosion.

Figure 5. Range of side slope conditions directly below the toe of properly constructed subbase layers of low drainability.



Looking South

Looking North

Figure 6. Foundation drainage condition is indicated by the presence or absence of subbase seepage water on the bicycle path. Both pictures were taken from the same location.

Figure 7. Wet condition of transverse cracks three days after a summer shower indicate the greater ability of a CRC pavement to move water to and from the base compared with conventional concrete pavements.

subbase water is removed principally as a result of evaporation over the entire side slope area. Thus, the concentration of salt at the toe of the subbase is reduced and its effects on vegetative cover attenuated according to the thickness and clay content of the material that covers the subbase layer.

The base in all cases appeared to be densely graded and relatively impervious. No readily observable differences in base drainage characteristics were obvious within each project or from test section to test section.

The subgrade is typically a stiff to very stiff, dense, stable silty clay. The high support capacity of the clayey subgrade precludes it from being a factor in the occurrence of longitudinal cracking. Where sandy subgrades occurred, the subbase was, by observation, in a better drained condition than adjacent subbase on clayey subgrade. The clayey subgrade of Test Sections 1 and 2 was notably softer than the subgrade of the other test sections.

During field testing, it was observed that complete subbase saturation could occur as a result of a brief shower and that days later the subbase remained saturated. Therefore, the subbase, in at least some areas, can sorb water faster than the water can be drained away. It was also found that the subbase of a portion of Test Section 3 was much drier than the subbase of the other test sections. The dry subbase starts about half way up a long, vertical grade, shown in the view looking south in Figure 6. Looking north from the same point, the wet subbase condition is reflected by the wet condition of the bicycle path shown in Figure 6. It was found that while constructing the bicycle path, excessive side slope seepage was corrected by placing a drain directly under the path. The need and effectiveness of these drains are clearly illustrated in Figure 6.

Another observation was that transverse cracks in the CRC pavement act as wicks that feed water, by capillary action, to the base when the base is drier than the pavement surface. This process is reversed when the pavement surface is drier than the base. The wet condition found around transverse cracks for days after a summer rain (Fig. 7) indicates that the capillary size transverse cracks are removing water from the base layer. For this reason, transverse cracks should influence the moisture condition of the base layer. Movement of water by capillary size transverse cracks can be eliminated, essentially, by use of a base material having no capillary potential, i.e., material whose voids are too large to move water by capillary action. A change from dense-graded to open-graded base materials should prevent the detrimental movement of surface water to the base layer of CRC pavements.

TABLE 1
SUMMARY OF BASE MATERIAL PROPERTIES

Test Section No.	Sample Site No.	Thickness, in.	-200 Material, percent	Frost Susceptibility, mm/day	Permeability, ft/day
6	1	5.0	7.4	0.00	0.7
	2	4.8	7.5	0.39	0.2
	3	3.8	---	0.39	0.2
	4	3.5	7.8	--	0.02
	5	3.9	7.5	0.64	0.9
	6	4.3	7.2	0.00	0.02
	7	4.9	7.7	0.13	0.2
	8	4.5	8.3	0.26	0.05
	9	3.3	---	0.00	0.4
5	10	4.0	5.6	2.38	0.005
	11	3.8	5.5	2.15	---
	12	4.0	5.6	1.92	0.5
	12a	4.5	6.5	3.06	0.15
	13	4.2	5.0	1.07	0.007
	14	4.2	4.6	1.59	0.003
	14a	4.2	6.4	1.99	0.3
	14b	4.0	6.2	1.59	0.3
	15	3.8	5.2	1.19	0.2
	15a	3.8	7.9	2.03	*
	16	3.5	5.7	1.06	*
	17	4.5	2.6	2.04	1.8
	18	4.0	5.4	2.25	1.8
	19	6.2	3.7	1.82	---
4	20	4.0	---	1.72	0.4
	21	4.5	4.7	1.81	0.2
	22	4.3	4.6	1.63	0.9
	22a	4.3	4.1	1.77	0.005
	23	4.3	4.6	2.16	0.2
	24	4.3	6.9	2.33	0.2
	25	3.8	6.3	1.42	0.1
	26	3.8	6.9	2.16	0.6
	27	3.5	7.7	0.91	0.001
	28	4.3	4.4	1.02	0.005
	29	3.8	9.3	1.59	0.07
	29a	3.5	11.3	2.23	0.07
	30	4.0	9.2	1.64	0.05
3	31	4.0	8.8	3.28	*
	32	3.3	10.2	3.40	*
	33	3.5	9.3	4.07	*
	34	3.8	9.0	2.18	*
	35	3.8	8.2	2.70	*
	36	4.0	4.7	2.54	*
2	37	3.0	11.8	2.78	0.4
	38	4.3	8.4	1.43	0.6
	38a	3.8	10.4	1.06	0.7
	39	3.8	15.1	2.64	0.2
	40	4.0	16.9	2.11	0.09
	40a	4.0	9.6	1.50	0.5
	41	4.3	7.5	1.40	2.9
	42	3.8	12.9	1.20	3.3
	43	4.0	6.0	0.93	1.3
1	44	16.8	8.7	--	6.5
	45	18.3	6.6	--	4.9

* Indicates permeability of less than 0.1 ft/day, but actual value was not determined.

LABORATORY INVESTIGATION

Laboratory tests were conducted to determine if the pavement foundation is providing uniform pavement support. Non-uniform foundation support, in the transverse direction, could be a basic cause of the longitudinal cracking. Since the strength of granular foundation materials is affected primarily by drainage conditions and frost action, the laboratory study was concentrated on these characteristics of the base and subbase layers. Subgrade stability was investigated by relating in situ water content to Atterberg limits.

Drainage analysis consisted of determining the permeability and effective porosity of base and subbase layers (3). Because of the complex mixture of materials in the side slope area it was not possible to determine Casagrande's drainage time, t_{50}, of the subbase layer in place. When no water would drain through a test sample under constant head for a period of one hour the sample was classified as impervious. That is, its permeability is less than 0.1 ft/day, which is relatively impervious for base or subbase materials. For Test Sections 3 and 5, most base samples were found to be impervious. For the purpose of this report, all base samples that had not been previously tested or discarded were tested or retested using the falling head permeability test. Those data make it possible to examine the relative permeability of the three different types of base materials (i.e., steel furnace slag, limestone, and natural gravel) used on I 275.

Frost susceptibilities of base and subbase samples were determined using procedures described in Ref. (4).

Laboratory test results are summarized in Tables 1, 2, and 3 for base, subbase, and subgrade, respectively. In Table 4, the results are summarized based on permeability criteria established in Ref. (5), frost susceptibility criteria established in Ref. (3), and Atterberg limits criteria established by Casagrande (6).

Table 4 indicates that the permeability of the base layer for all test sections ranges from low to very low. In order to prevent accumulation of water in joints and cracks, to avoid pumping action, and to provide uniform support throughout the year, base permeability should be greater than 1,000 ft/day. The base is generally frost susceptible, ranging from negligible to medium with only Test Section 6, which has a natural aggregate base, in the negligible range.

Table 4 indicates that the subbase layers, for almost all of the sections have medium permeability. In order to provide uniform support throughout

TABLE 2
SUMMARY OF SUBBASE PROPERTIES

Test Section No.	Sample Site No.	Thickness, in.	-200 Material, percent	Percent Saturation At Tension, 2 to 3-in.	Permeability, ft/day, k	Effective Porosity, n_e	Dry Density, lb/cu ft	Drainage Restricted	
								Outside	Median Side
	1	11.5	3.3	---	0.1	---	126.5	Yes	Yes
	2	7.5	6.6	---	0.1	---	122.6	No	Yes
	3	15.0	4.7	100	1.0	0.0	155.6	No*	Yes*
	4	14.0	6.0	96	0.7	0.01	117.2	No*	Yes*
6	5	15.3	9.7	90	19.1	0.03	113.5	No*	No*
	6	10.0	8.4	64	16.6	0.01	118.6	Yes*	No*
	7	9.5	8.6	---	0.1	---	114.9	No*	No*
	8	11.5	15.8	---	0.1	---	113.7	Yes*	No*
	9	13.5	9.7	---	11.4	---	131.6	No*	Yes*
	10	13.5	7.5	86	17.4	0.04	108.0	No	No
	11	13.0	5.0	93	15.1	0.02	109.2	No	No
	12	10.8	3.5	94	19.2	0.02	109.9	Yes	Yes
	12a	13.5	10.8	94	3.4	0.03	111.8	Yes	No
	13	11.5	14.7	89	5.0	0.03	114.2	Yes	No
	14	11.0	12.4	---	--	---	111.8	Yes	Yes
	14a	10.0	13.6	100	9.0	0.00	114.9	Yes	Yes
5	14b	9.0	6.9	81	31.9	0.06	111.8	Yes	No
	15	11.0	11.1	97	6.1	0.01	111.8	No	Yes
	15a	11.5	5.9	90	39.1	0.04	108.0	Yes	Yes
	16	12.5	6.4	85	8.2	0.05	110.5	Yes	Yes
	17	10.2	3.3	92	19.8	0.03	107.4	Yes	Yes
	18	10.2	2.6	95	14.1	0.01	111.1	No	Yes
	19	11.8	4.1	84	14.7	0.06	113.0	Yes	Yes
	20	11.0	3.5	87	13.0	0.03	112.9	Yes	Yes
	21	12.8	2.4	100	20.0	0.00	113.7	Yes	Yes
	22	11.8	4.8	87	5.0	0.04	112.5	Yes	Yes
	22a	10.8	4.4	97	18.0	0.008	115.8	Yes	Yes
	23	9.3	3.8	85	14.0	0.05	111.9	Yes	Yes
	24	11.0	3.3	89	17.0	0.03	112.6	Yes	Yes
4	25	11.8	5.3	89	24.0	0.04	112.0	Yes	Yes
	26	11.8	2.4	91	34.0	0.03	107.9	Yes*	Yes*
	27	12.3	3.0	90	16.0	0.03	112.7	No*	No*
	28	11.8	4.9	83	4.6	0.06	108.4	Yes	No
	29	12.0	6.3	98	13.6	0.006	113.6	No	No
	29a	16.0	11.4	85	2.1	0.05	112.5	Yes	Yes
	30	11.0	8.5	96	5.6	0.01	118.6	Yes	No
	31	12.0	8.0	88	13.7	0.04	108.6	Yes	No
	32	10.0	22.1	94	3.6	0.02	113.6	Yes	Yes
3	33	12.5	11.1	97	3.3	0.02	113.0	No	No
	34	14.0	8.6	95	7.0	0.01	112.4	No	Yes
	35	11.0	7.8	93	12.3	0.02	114.2	Yes	Yes
	36	9.5	8.7	92	8.0	0.02	110.5	Yes	No
	37	13.8	4.2	100	8.8	---	112.0	Yes	Yes
	38	12.0	5.2	93	0.5	0.02	111.4	Yes	Yes
	38a	11.5	5.5	100	2.9	---	114.5	Yes	Yes
	39	11.3	5.5	82	2.0	0.06	109.1	Yes	Yes
2	40	13.0	4.0	100	1.1	---	113.0	Yes	Yes
	40a	15.3	5.7	100	4.9	---	110.3	Yes	Yes
	41	12.5	5.6	100	12.3	---	110.3	Yes	Yes
	42	12.5	4.3	94	6.8	0.02	107.2	Yes	Yes
	43	12.0	4.9	97	3.3	0.007	111.3	Yes	Yes
1	44	No Subbase Layer							
	45	No Subbase Layer							

* Indicates permeability of less than 0.1 ft/day, but actual value was not determined.

TABLE 3
SUMMARY OF SUBGRADE PROPERTIES

Test Section No.	Sample Site No.	Plasticity Index	Plastic Limit	Liquid Limit	Moisture Content, percent	-200 Material, percent	AASHTO Classification
6	1	11	14	25	11	68	A-6
	2	4	11	15	12	51	A-4
	3	9	14	23	15	71	A-4
	4	--	--	--	--	--	--
	5	N.P.	--	--	14	26	A-2-4
	6	N.P.	--	--	13	14	A-2-4
	7	--	--	--	11	--	--
	8	N.P.	--	--	12	17	A-2-4
	9	N.P.	--	--	12	28	A-2-4
5	10	N.P.	--	--	--	38	A-4
	11	N.P.	--	--	--	19	A-2-4
	12	N.P.	--	--	--	17	A-2-4
	12a	10	14	24	12	68	A-4
	13	10	14	24	10	72	A-4
	14	11	16	27	13	68	A-6
	14a	11	15	26	12	68	A-6
	14b	11	15	26	13	65	A-6
	15	12	15	27	12	71	A-6
	15a	11	14	25	12	67	A-6
	16	12	14	26	11	69	A-6
	17	10	14	24	9	64	A-4
	18	13	17	30	15	84	A-6
	19	12	14	26	13	78	A-6
4	20	11	15	26	11	80	A-6
	21	10	14	24	11	69	A-4
	22	18	20	38	18	73	A-6
	22a	18	18	36	20	75	A-6
	23	13	15	28	14	65	A-6
	24	10	15	25	11	69	A-4
	25	10	14	24	10	68	A-4
	26	N.P.	--	--	9	15	A-2-4
	27	N.P.	--	--	10	8	A-3
	28	14	16	30	16	70	A-6
	29	12	14	26	11	68	A-6
	29a	--	--	--	--	--	--
	30	11	13	24	8	63	A-6
3	31	13	14	27	10	70	A-6
	32	14	16	30	12	72	A-6
	33	20	20	40	17	78	A-6
	34	13	16	29	12	67	A-6
	35	16	19	35	16	87	A-6
	36	19	20	39	17	88	A-6
2	37	--	--	--	--	--	--
	38	19	20	39	18	78	A-6
	38a	14	16	30	27	70	A-6
	39	13	15	28	28	66	A-6
	40	12	15	27	12	68	A-6
	40a	12	15	27	14	66	A-6
	41	11	13	24	14	63	A-6
	42	17	22	39	24	76	A-6
	43	9	13	22	16	62	A-4
1	44	10	14	24	16	68	A-4
	45	11	15	26	17	67	A-6

TABLE 4

CONDENSED BASE, SUBBASE AND SUBGRADE PROPERTIES

Test Section No.	Sample Site No.	Base Permeability	Base Frost Susceptibility	Subbase Permeability	Subgrade Consistency
6	1	Low	Negligible	Very low	Very stiff
	2	Low	Negligible	Very low	Stiff
	3	Low	Negligible	Low	Stiff
	4	Low	Negligible	Low	---
	5	Low	Negligible	Medium	Non-plastic
	6	Low	Negligible	Medium	Non-plastic
	7	Low	Negligible	Very low	Non-plastic
	8	Low	Negligible	Very low	Non-plastic
	9	Low	Negligible	Medium	Non-plastic
5	10	Very low	Medium	Medium	Very stiff
	11	---	Medium	Medium	Very stiff
	12	Low	Low	Medium	Very stiff
	12a	Low	Medium	Medium	Very stiff
	13	Very low	Low	Medium	Very stiff
	14	Very low	Low	Medium	Very stiff
	14a	Low	Low	Medium	Very stiff
	14b	Low	Low	Medium	Very stiff
	15	Low	Low	Medium	Very stiff
	15a	Low	Medium	Medium	Very stiff
	16	Low	Low	Medium	Very stiff
	17	Low	Medium	Medium	Very stiff
	18	Low	Medium	Medium	Very stiff
	19	Low	Low	Medium	Very stiff
4	20	Low	Low	Medium	Very stiff
	21	Low	Low	Medium	Very stiff
	22	Low	Low	Medium	Very stiff
	22a	Very low	Low	Medium	Very stiff
	23	Low	Medium	Medium	Very stiff
	24	Low	Medium	Medium	Very stiff
	25	Low	Low	Medium	Very stiff
	26	Low	Medium	Medium	Non-plastic
	27	Very low	Very low	Medium	Non-plastic
	28	Very low	Low	Medium	Very stiff
	29	Low	Low	Medium	Very stiff
	29a	Low	Medium	Medium	---
	30	Low	Low	Medium	Very stiff
3	31	Low	Medium	Medium	Very stiff
	32	Low	Medium	Medium	Very stiff
	33	Low	High	Medium	Very stiff
	34	Low	Medium	Medium	Very stiff
	35	Low	Medium	Medium	Very stiff
	36	Low	Medium	Medium	Very stiff
2	37	Low	Medium	Medium	---
	38	Low	Low	Low	Very stiff
	38a	Low	Low	Medium	Very soft
	39	Low	Medium	Low	Very soft
	40	Low	Medium	Low	Very stiff
	40a	Low	Low	Medium	Very stiff
	41	Medium	Low	Medium	Stiff
	42	Medium	Low	Medium	Medium
	43	Low	Very low	Medium	Stiff
1	44	Medium	---	---	---
	45	Medium	---	---	---

the year and to avoid pumping into open-graded base layers, the subbase should, it is thought, meet Casagrande's drainage criteria (7), i.e., the permeability values should be in the medium to high range and should not exceed 90 percent saturation when gravity drained at 2 to 3 in. of tension. The data in Table 2 indicate that while the subbase, generally, has adequate permeability, it frequently is over 90 percent saturated when drained. Therefore, it can be subject to volume change on freezing resulting in a loss of support capacity when thawing. Under these conditions, wheel loads can cause an increase in tensile stresses in the CRC slab greater than the stress generated by the same wheel loads after the foundation has reconsolidated.

Table 3 indicates that for Test Sections 3 to 6, the subgrade is either granular or a silty clay in very stiff, solid condition (in situ water content below the plastic limit). For Test Sections 1 and 2, the subgrade is in a stiff or semi-solid condition (in situ water content above the plastic limit). Although the subgrade of Test Sections 1 and 2 is generally not as strong as that of the other sections, it is, nevertheless, in a good, stable condition. In addition, on the basis of Casagrande's Plasticity Chart, the cohesive subgrade soils are classified as having low plasticity.

DISCUSSION

Subgrade

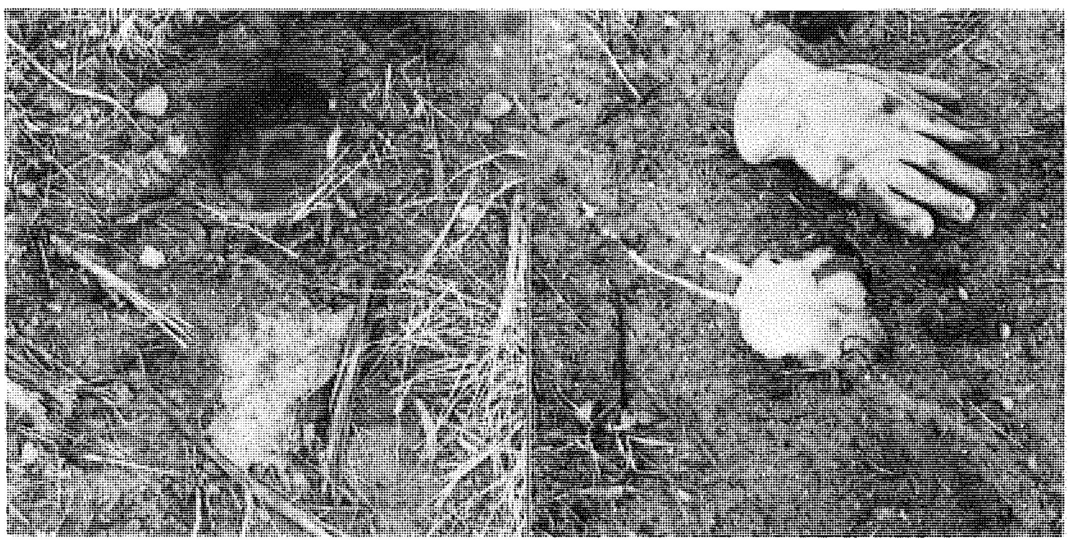
The field and laboratory studies indicate that the subgrade is in good to excellent condition and is providing uniform support for the CRC pavement. Therefore, it is concluded that the subgrade's performance to date is very good and is not related to the occurrence of longitudinal cracks.

Subbase

The field study indicated that, generally, the subbase layer would become completely saturated as a result of a brief summer rain and that days later the subbase would still be completely saturated. Laboratory results support this observation. The moderate permeability and high capillary potential of the subbase permits rapid sorption of water and this along with side slope restrictions tend to account for slow subbase drainage. However, it is not known how surface infiltrating water could get into the subbase so rapidly, particularly when the base is so impervious.

From a structural standpoint, the subbase will be seasonally weakened as a result of volume increases caused by frost action. As the subbase thaws, it is subject to consolidation under traffic loading. Settlement that occurs as the subbase layer reconsolidates is time dependent because of hydrodynamic time lag and secondary compression. On thawing, the full traffic and overburden pressures are carried completely by the pore water. As the weight of the pavement and traffic causes the subbase to consolidate, pavement loads are slowly transferred from pore water to the subbase sand particles. The subbase is in a weakened condition from the time thaw first occurs until all excess pore pressures are completely dissipated. It is during this period of reconsolidation that longitudinal tensile stresses at the center of each lane can exceed the stresses developed after reconsolidation.

On April 2, 1979, auger borings were made in the side slope areas to investigate the saturation condition of the subbase layer soon after thawing. At all sites investigated, the free water level was at or near the surface of the side slope. However, two distinctly different pore water conditions were found as shown in Figure 8. Free water in some auger holes was, essentially, unaffected by traffic load, i.e., the water did not surge about and develop a foam as trucks passed by. It could not be determined if this was due to the fact that no excess pore water pressure was generated by the traffic load, or to the permeability of the subbase, including blockage in the side slope area, being so restrictive as to cause considerable hydrodynamic time lag and hence, exert no effect on pore water pressure at this



Light foam indicates low excess pore water pressure.

Heavy foam indicates high excess pore water pressure.

Figure 8. Foam develops when traffic-generated excess pore water pressure causes a surging action in free water found in holes augered into the subbase layer between the edge of shoulder and the toe of the subbase.

distance from the load. By contrast, at other sites, the passing of trucks developed such a strong surging action that a foam quickly developed on the free water surface. This condition indicated that several weeks after complete subbase thaw, excess pore water pressures are being generated by traffic loads for a distance of 15 ft or more from the edge of the pavement.

It has been concluded that the sand subbase on the portion of I 275 studied will, in most cases, be subject to seasonal weakening which is in several ways accelerating the rate of deterioration of the pavement. The volume change characteristic of the subbase, caused by frost action, results, when thawing, in increased stressing of the pavement during subbase reconsolidation. This can cause, or contribute to, longitudinal cracking. Excess pore pressure during subbase reconsolidation is causing lateral migration of subbase material which may contribute to shoulder heave and loss of subbase material such as that shown in the upper left corner of Figure 5. Both these factors could cause the portion of the pavement subjected to the heaviest traffic to subside which, in turn, would cause the adjacent slab and shoulder to rise. This action could also increase transverse tensile stresses at the center of each lane. Once longitudinal cracks occur and in the absence of transverse steel, subbase deficiencies could have a significant detrimental effect on the pavement's performance, mostly in the form of vertical displacements of the pavement surface and in additional longitudinal cracking that can result in punch-outs.

Not all of the subbase problems on I 275 are caused by side slope blockage and the covering of side slopes with dirt and sod. As shown in Table 2, there are areas of the subbase that have a high capillary potential, indicated by the high percentage saturation (over 90 percent) at 2 to 3 in. of tension. These areas cannot be drained sufficiently by gravity action to prevent possible frost heave when frozen, and the development of excess pore water pressure when thawing. In cases where the subbase will be over 90 percent saturated with capillary water when gravity drainage is complete, retrofitting subbase drains would not prevent frost action and reconsolidation problems from occurring. Such drains will, however, attenuate the damage by providing more rapid removal of surface water that enters the 'eavestrough joint' at the pavement shoulder interface and by increasing the rate of consolidation, which will reduce the time the pavement is in a weakened condition.

Identification of subbase materials having high capillary potential can be made using field procedures outlined in Ref. (8). Suitable subbase materials will allow gravity drainage to lower the water content of the subbase layer to less than 90 percent saturation. Materials having high capillary potential can be improved by increasing moisture tension, i.e., increasing the thickness of the subbase layer. Because of undercutting, thicker subbase layers occurred in portions of Sections 4, 5, and 6, sample sites 6 to 12 and 26 and 27, and these areas are presently free of longitudinal cracking. Increased subbase thickness requirements can be readily determined using standard laboratory moisture tension test procedures. Subbase layers that are less than 90 percent saturated when gravity drained, but which drain too slowly to avoid problems with volume change on freezing, can be corrected by the addition of supplemental subbase drains. The procedures outlined in Ref. (9) enable computation of the supplemental subbase drain spacing necessary to reduce drainage time to acceptable limits.

Base

Depending upon the test section, steel furnace slag, limestone, and natural gravel materials were used for the base layer. The limestone has low frost susceptibility, the slag has moderate frost susceptibility, and the natural aggregate has negligible frost susceptibility. As shown in Figure 9, longitudinal cracking occurs in sections with all three types of base material.

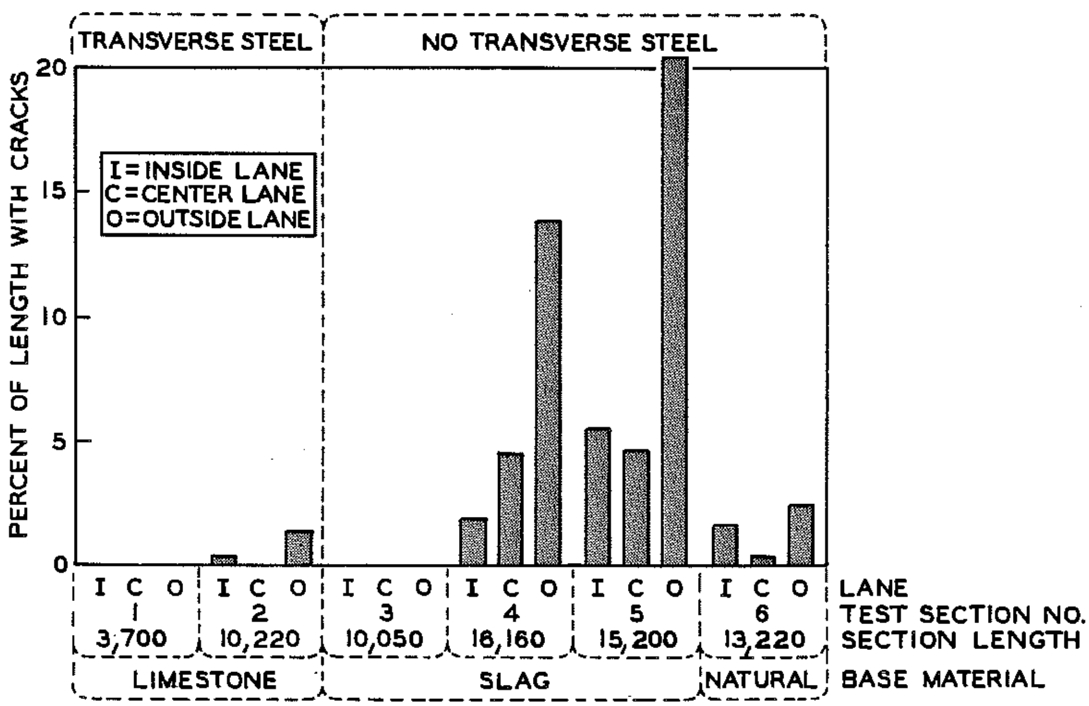


Figure 9. Percent of lane with longitudinal cracks on test sections 1 through 6 as of March 1979.

Section 1 is supported by an open-graded limestone base which is about 14 in. thick. The top 3 to 4 in. of the base is more densely graded than is the rest of the base. A base such as this should not frost heave or develop excess pore pressure, even when drainage is restricted at the side slope, as shown in Figure 3. Section 1 should have good foundation support and it is uncracked. Section 2 has a dense-graded limestone base, equally as frost susceptible as the bases of the more heavily cracked Sections 4 and 5, yet it is relatively uncracked and those cracks that do exist are very tight, indicating the effectiveness of transverse steel in preventing cracks from developing into more serious forms of failure. The base of Section 3 is moderately to highly frost susceptible and the pavement has no longitudinal cracking. The good performance of Section 3 is contrary to expectations based on its base properties. The magnitude and extent of shoulder heave in Section 3 indicates that it should be longitudinally cracked. Sections 4 and 5 are the most heavily cracked and the frost susceptibility characteristic of the base for Section 6 would also indicate that it should be uncracked if frost action in the base were the cause of longitudinal cracking. Because of the lack of association of longitudinal cracking and base frost susceptibility, it would appear as if base frost action is not related to longitudinal cracking. However, it is suggested that a special form of frost action is causing some areas of pavement to heave at the pavement-shoulder joint and that this heave, which is not directly related to frost susceptibility, is primarily responsible for most of the longitudinal cracking.

There may be some question as to why slag bases are significantly more frost susceptible than either the limestone or natural gravel bases, although they generally have lower fines content and good dense gradation characteristics. The reason is that soils, even sands, containing hydroxides of iron, manganese, and aluminum show great capillarity and comparatively high osmotic pressures which produce a corresponding lowering of the freezing point (10). The lowering of the freezing point around the hydroxide particles facilitates water transport along particle surfaces, thus continuous water absorption into the frost zone is assumed, leading to the formation of ice layers and ice lenses. Further, the addition of sulfates to slag, in the form of acid treatment, increases its water holding capacity by decreasing the vapor pressure of the water contained. These characteristics of dense-graded slag indicate its undesirable nature for use as base course material. Coarse aggregate steel slag meeting 6A, 9A, or 17A grading requirements may, essentially, eliminate the undesirable properties of dense gradations. The Department has discontinued use of dense graded steel furnace slag.

Shoulders

The shoulders, as well as the pavement, are subject to frost heave. Since shoulder thickness at the pavement-shoulder interface is equal to that of the pavement, no differential heave was expected. Shoulders, however, are often found to have heaved considerably more than the pavement. In addition, the shoulder does not return to its original position by mid-summer (Fig. 10). The reason for this is that while the pavement and shoulder are subject to equal heave potential, the greater vertical stress on the pavement, caused by traffic, accelerates the rate of reconsolidation of the base and subbase layer under the pavement compared to that under the shoulder.

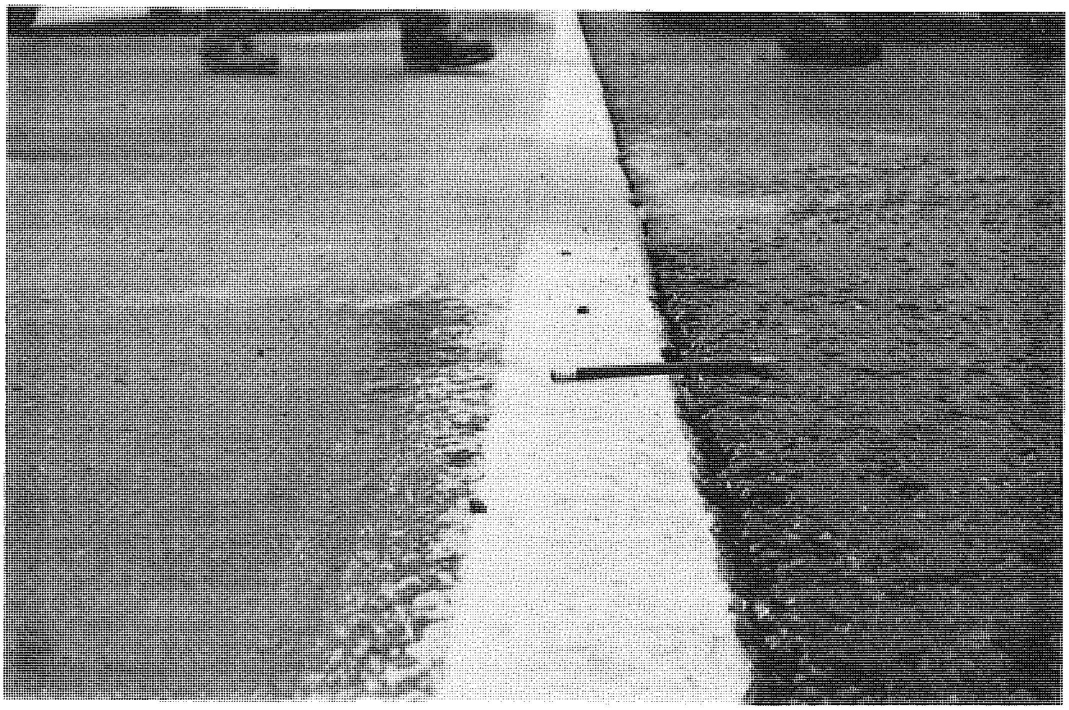


Figure 10. Permanent vertical displacement of the shoulder caused by frost action in the base layer.

Lateral displacement of base material from pavement to shoulder area and vertical shoulder pressures, too small to completely reconsolidate the base under the shoulder, are thought to be the reasons for the apparent permanent upward shoulder displacement.

Frost Tenting Phenomenon

Two forms of frost heave are measured by the test method used in this study. The most common and severe form occurs when the capillary potential of the soil is so high that water is freely fed, by capillary action, to the freezing zone where ice lenses form and cause expansion or frost heave. A second form of heave is caused entirely by expansion of water on freezing, under conditions where the percent saturation exceeds 90 percent. In this case the movement of water is so restricted that there is essentially no change in water content on freezing. Both these forms of frost heave should act transversely across the pavement in a uniform manner and, therefore, should not be responsible for any differential support that could initiate longitudinal cracking. Base and subbase deficiencies are similar, but because the pavement rests directly upon the base, base characteristics should have the stronger detrimental influence on pavement performance.

This study indicates that a special form of frost heave may be responsible for the differential support characteristic necessary to initiate longitudinal cracking. Differential frost heave is thought to occur along cracks and joints, such as the longitudinal pavement-shoulder joint, which can supply saltwater to the freezing zone. This kind of frost action works in reverse of normal frost action in that the freezing zone moves upward and toward the source of saltwater. In normal frost action, freezing takes place along an essentially horizontal plane that moves downward toward the source of free water whereas in this special form of frost action, the freezing zone is semicircular and moves upward toward the source of saltwater. An excellent detailed explanation of this phenomenon is presented in Ref. (11). This form of frost action has been associated with transverse cracks of flexible pavement and has been found at undowelled joints in rigid pavement. The condition is commonly referred to as 'frost tenting.'

In order for the edge of the pavement and shoulder to be pushed upward by frost tenting action, five conditions are reported to be necessary (11).

1) The base must be completely capillary saturated prior to or during the freezing period,

2) The pavement crack must provide a free supply of water,

3) The base must contain at least 3 percent of soil grains finer than 0.02 mm,

4) A gradual decrease in subfreezing temperature of the pavement surface must take place, and

5) The water entering the pavement cracks must contain salt.

Any base thaw will result in an enlargement of the salt saturated zone so that, depending on weather conditions, the entire base and subbase may eventually become salt saturated. When this happens, frost tenting action can no longer take place. The most severe differential frost heave occurs when the frost tenting action is confined to a relatively small area around a joint or crack. As weather conditions permit, larger cross-sectional

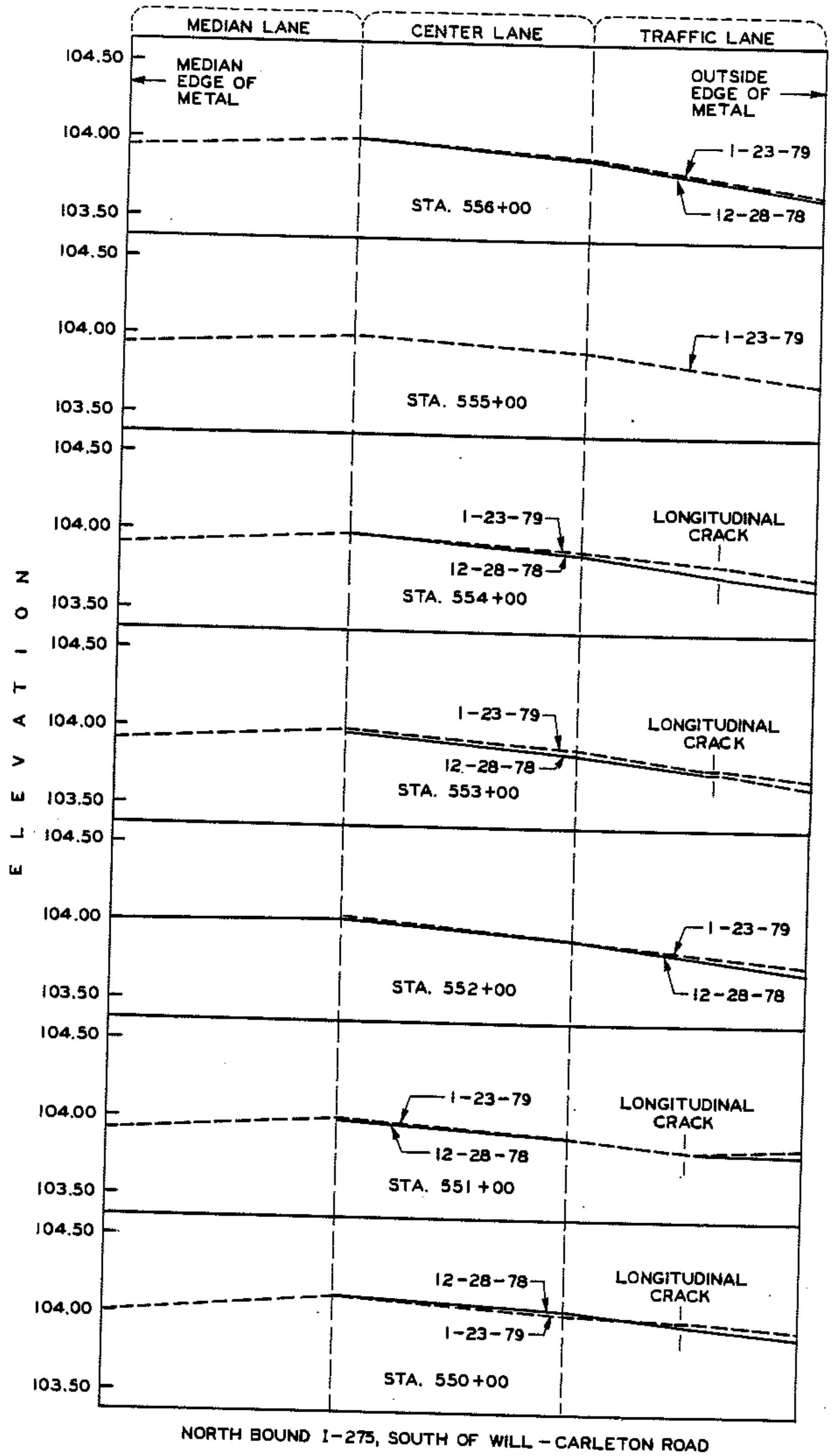


Figure 11. Transverse pavement profile at Sta. 552+00, 551+00, 550+00 indicating a tendency for the outside edge of metal to heave more than the rest of the pavement.

areas of base and subbase become salt saturated, and the severity of any differential heave from frost tenting action diminishes. Therefore, during a mild winter when salt is applied to the pavement during early wet snows, the base and subbase can become completely salt saturated prior to freezing. Under such conditions, there should be little tendency for frost tenting to occur. A very cold, dry winter initially, in which the base and subbase freeze prior to the use of salt for ice control would, on the other hand, create ideal conditions for frost tenting action, provided salt applications become necessary and the temperature remains below freezing. During the 1980 winter surveys on I 275, the pavement edge at the longitudinal pavement-shoulder joint heaved, in general, no more than did the rest of the pavement. This is believed to be due to the mild winter in which precipitation was below normal and temperatures, though near normal, rarely were as low as needed for frost tenting to occur. For this reason, it is concluded that the occurrence of new longitudinal cracks between 1979 and 1980 surveys would be minimal or non-existent. However, detrimental base and subbase properties are such that existing longitudinal cracks may continue to increase in length. The additional cracking that occurred between 1979 and 1980 has been described and summarized in an office memorandum (12) which appears to confirm this conclusion.

The 1978-79 freezing season was exceptionally cold and snowy. A limited survey of pavement elevations during the 1978-79 freezing season indicated a tendency for the pavement edge to heave more than the rest of the pavement (Fig. 11). This tendency, however, as Figure 12 illustrates, did not exist at all sites. The longitudinal cracks shown in Figure 13 indicate there is a logical relationship between frost tenting action at the pavement-shoulder edge and longitudinal cracking. However, frost tenting action does not seem to explain center lane longitudinal cracking, such as that shown in Figure 14. It is concluded that because a majority of the longitudinal cracking occurs in the lane on the low side of the superelevation, most longitudinal cracking is related to frost tenting action at the pavement-shoulder joint. Since center lane longitudinal cracking cannot be directly attributed to frost tenting action, it is concluded that development of excess pore water pressure in the base and subbase layers, induced by traffic loads as previously discussed, is a prime causal factor in the occurrence of longitudinal cracking in general, and may be the sole cause of longitudinal cracking in the case of center lane longitudinal cracking.

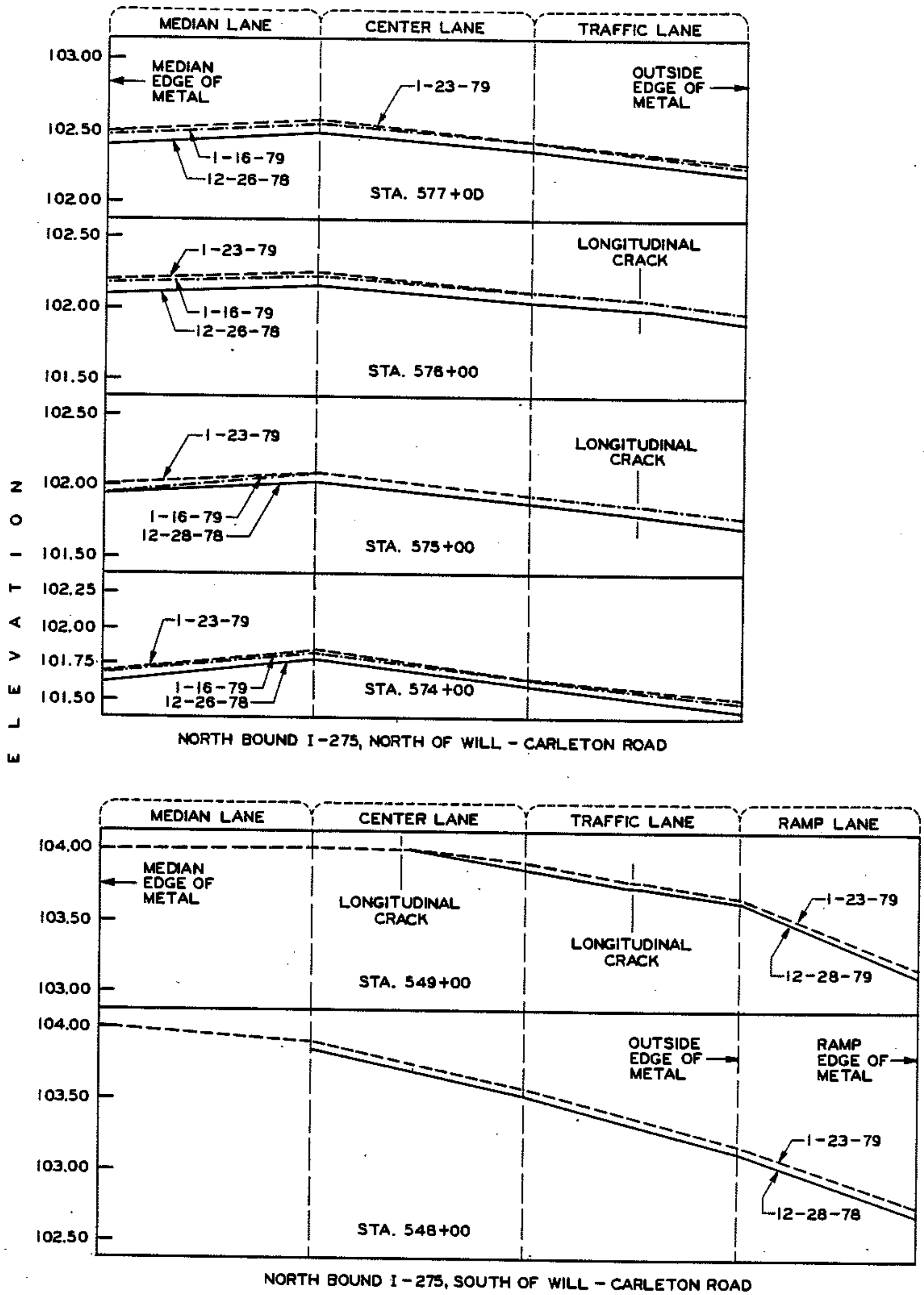


Figure 12. Transverse pavement profile at Sta. 577+00, 576+00, 575+00, 574+00 indicating no edge of metal tendency for the outside edge of metal to heave more than the rest of the pavement.

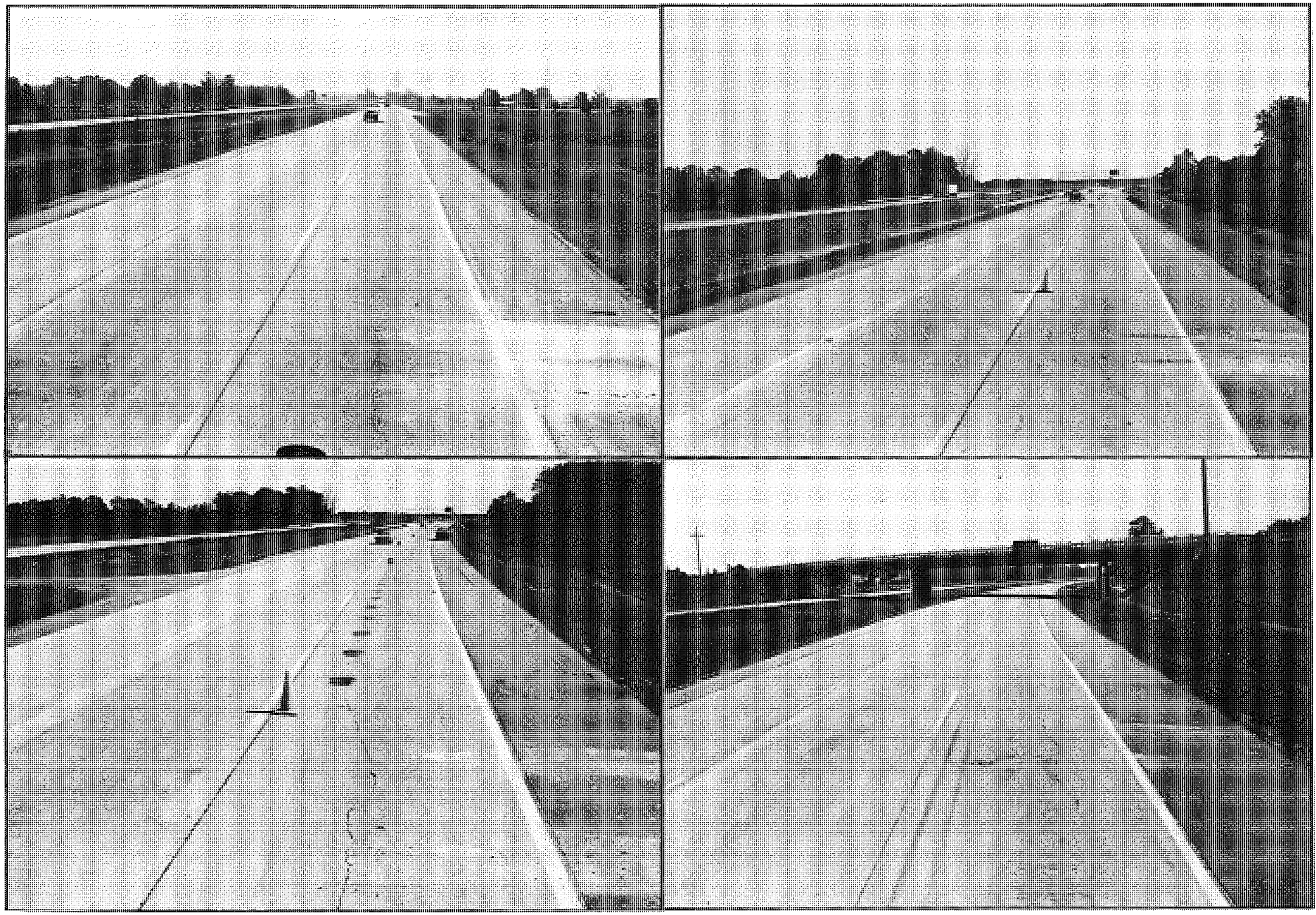


Figure 13. Longitudinal cracking which could be caused by 'frost tenting' action of the base at the longitudinal pavement-shoulder joint.

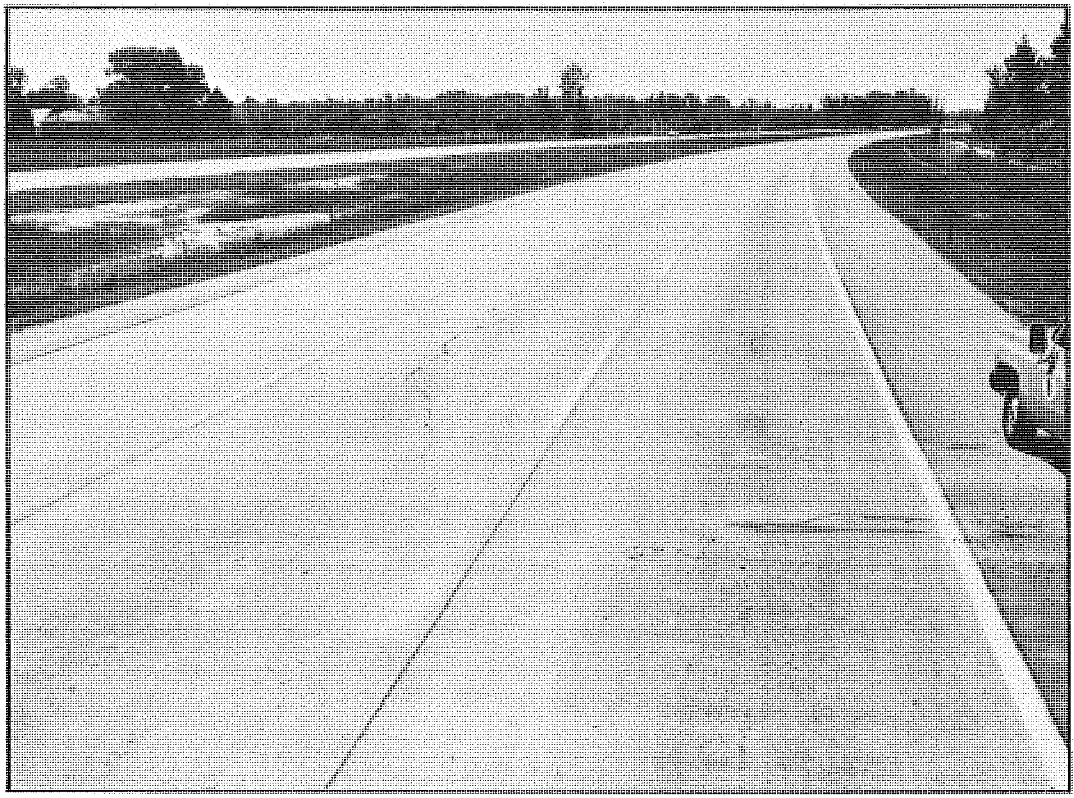


Figure 14. Longitudinal cracking not related to 'frost tenting' action.

CONCLUSIONS

The foundation investigation of I 275 indicated the subgrade to be well constructed, strong, and of stable seasonal strength. The subbase layer on the other hand, has some problems. This investigation indicates there is a need to expand subbase design methodology to include gravity and capillary drainability computations if this layer is to provide better year-long pavement support capacity and protect the subgrade from detrimental environmental actions. Subbase material specifications and acceptance test procedures should be modified to ensure adequate subbase drainage capacity and to enable rejection of materials that cannot be drained by gravity action. Such acceptance test procedures have been developed and successfully undergone trial field use. The periodic loss of subbase support capacity, because of migration of fines and excess pore water pressures, may influence the occurrence of new longitudinal cracks or cause the extension of existing longitudinal cracks.

Differential frost heave in the base layer at the pavement-shoulder interface, due to 'frost tenting' action, is believed to be responsible for the occurrence of most of the longitudinal cracking. Such differential frost heave requires a critical combination of an abundant supply of surface saltwater, temperatures low enough to cause freezing of saltwater, and the proper combination of low permeability and base frost susceptibility. The absence of any one of these factors should prevent the occurrence of differential heave. In conducting winter studies of I 275 during the 1979-80 freezing season, it was noted that no significant differential frost heave occurred. This indicates that climatic conditions during the 1979-80 freezing season were not conducive to differential frost heave. However, extension of existing longitudinal cracks may occur as a result of the seasonally weaker base-subbase condition.

The attenuation of further longitudinal cracking may be accomplished by the installation of a specially designed drain under the lower pavement-shoulder interface which would remove all surface infiltrating saltwater, prevent the base from sorbing surface run-off saltwater, and provide positive subbase drainage. By depriving the base of its source of saltwater, the potential for differential frost heave at the low edge of the pavement should be removed and the occurrence of new longitudinal cracks attenuated. Pavement areas in need of drains may be identified as those areas where permanent shoulder displacement occurs. Positive subbase drainage should help attenuate extension of existing longitudinal cracks in those areas where the subbase is gravity drainable. However, the high capillary potential nature of the base and much of the subbase materials is such that longitudinal cracking could continue because of migration of fines and excessive pore water pressures, even with the installation of drains.

Standard edge drains have been installed at the shoulder-pavement interface of Test Sections 4 and 5. Performance of these test sections is being closely monitored. In addition, standard edge drains have been installed in Control Section 82292, Job No. 17606A. Within this control section, the specially designed drain shown in Figure 15 was installed in the northbound lane only and extends from Station 1142+00 to 1167+34.

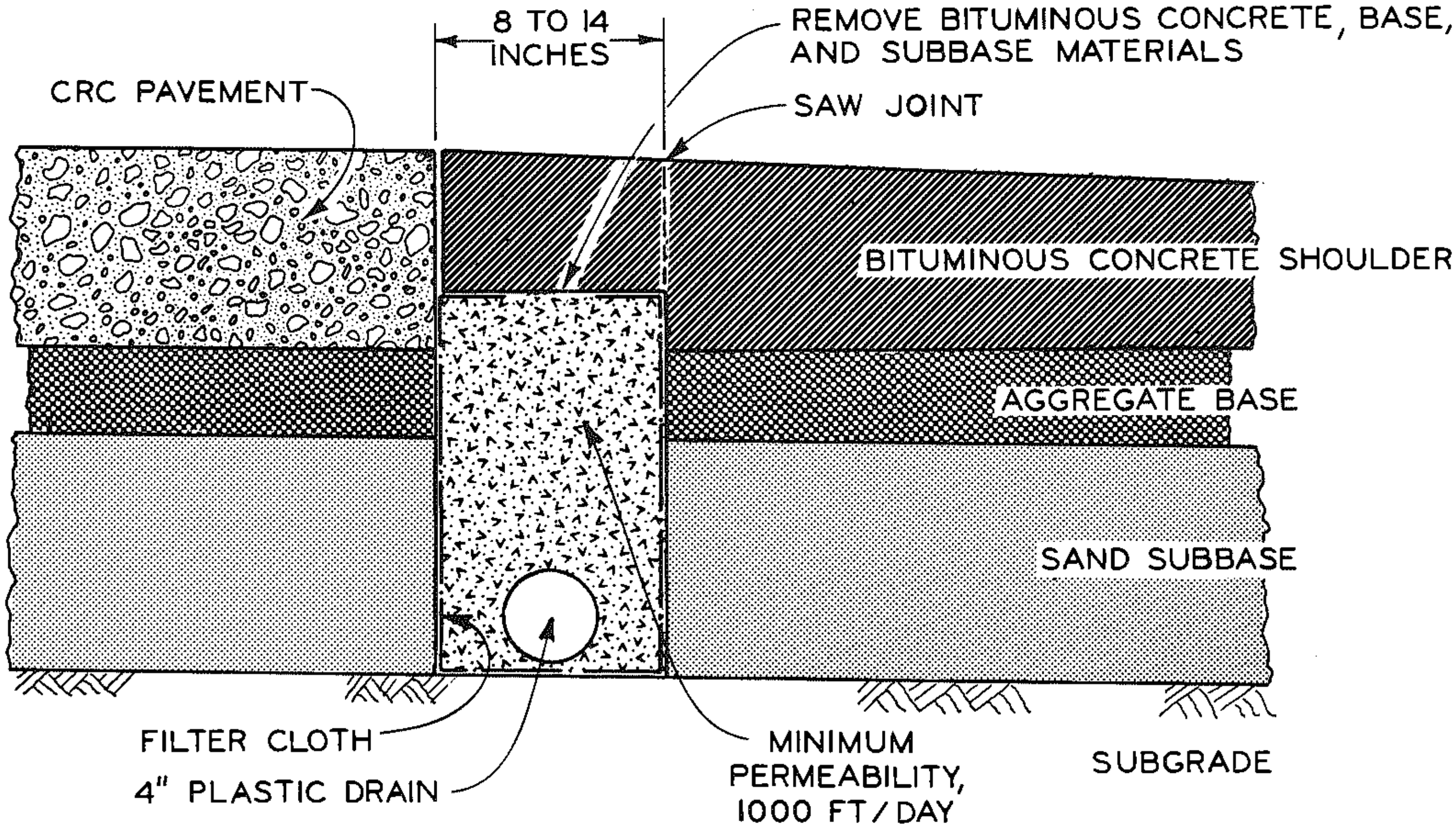


Figure 15. Typical drain cross-section.

REFERENCES

1. Investigation of the Foundation Condition of I 275, Research Project 73 F-129, Departmental correspondence from E. C. Novak, Jr. to G. R. Cudney, February 15, 1979.

2. "Investigation of Pavement Problems on I 275," MDOT Research Report No. R-1126, October 1979.

3. Nettles, E. H. and Calhoun, C. C., Jr., "Drainage Characteristics of Base Course Materials, Laboratory Investigation," U. S. Army Engineers Waterways Experiment Station, Corps of Engineers Technical Report No. 3-786, July 1967.

4. Kaplan, C. W., "A Laboratory Freeze Test to Determine the Relative Susceptibility of Soils," U. S. Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire, May 1965.

5. Lambe, T. W. and Whitman, R. V., "Soil Mechanics," John Wiley and Sons, Inc., 1969.

6. Peck, R. B., Hanson, W. E., and Thornburn, T. H., "Foundation Engineering," John Wiley and Sons, Inc., 1954.

7. Casagrande, A. and Shannon, W. L., "Base Course Drainage for Airport Pavements," Proceedings ASCE, Vol. 77, Separate No. 75, June 1951.

8. Novak, E. C., Jr., "Development and Evaluation of a Field Drainability Test Method," MDOT Research Report No. R-1001, 1976.

9. Novak, E. C., Jr., "A Method of Determining Subbase Drainability," MDOT Research Report No. R-805, April 1972.

10. Ducker, A., "Is There a Dividing Line Between Non-Frost Susceptible and Frost-Susceptible Soils?" National Research Council of Canada, Technical Translation 722, 1958.

11. Shober, S., "Frost Tenting," Wisconsin Department of Transportation, June 1971.

12. Office memorandum to K. A. Allemeier from L. T. Oehler, "Results of 1980 Longitudinal Crack and Punch-Out Survey on I 275," Research Project 79 F-158, July 14, 1980.