State of Illinois
DEPARTMENT OF TRANSPORTATION
Bureau of Materials and Physical Research

EFFECT OF SUBBASE TYPE AND SUBSURFACE DRAINAGE ON BEHAVIOR OF CRC PAVEMENTS

By

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Interim Report
IHR-36
Investigation of Continuously Reinforced Concrete Pavement

A Research Project Conducted by
Illinois Department of Transportation
in Cooperation with
U. S. Department of Transportation
Federal Highway Administration

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May 1979

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/IL/PR-083		

4. Title and Subtitle
Effect of Subbase Type and Subsurface Drainage on Behavior of CRC Pavements

5. Report Date
May 1979

6. Performing Organization Code

7. Author(s)
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8. Performing Organization Report No.
Physical Research No. 83

9. Performing Organization Name and Address
Illinois Department of Transportation
Bureau of Materials and Physical Research
Springfield, Illinois 62706

10. Work Unit No.

11. Contract or Grant No.
IHR-36

12. Sponsoring Agency Name and Address
Illinois Department of Transportation
Bureau of Materials and Physical Research
126 East Ash Street
Springfield, Illinois 62706

13. Type of Report and Period Covered
Interim
May 1971 - December 1977

14. Sponsoring Agency Code

15. Supplementary Notes
Study Title: IHR-36 - Investigation of Continuously Reinforced Concrete Pavement. Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration

16. Abstract
As part of an extensive study conducted to determine the significant relationship between the behavior of CRC pavement and various design features, an evaluation was made of four types of subbases and three types of subsurface drainage systems. Deformation of the subbase due to loading and/or subbase erosion under severe climate and moisture conditions can cause severe pavement distress. To prevent or delay these types of pavement distress from developing early in the service life of the pavement, some type of stable subbase and subsurface drainage system is necessary.

The pavement behavior, expressed in terms of transverse cracking and deflections, was analyzed and correlated with the type of subbase and type of subsurface drainage system. The investigation also includes the procedure used to evaluate the efficiency of the drainage systems and to expand the knowledge of the stress levels in the steel.

The pavement sections containing the various subbases and subsurface drainage systems are performing excellently after 6½ years of service. The lime-stabilized soil mixture as subbase offers the potential for reduced construction costs, and it is recommended that additional sections be built for further evaluation. The subsurface drainage system with longitudinal underdrains placed at the edge of the stabilized subbase was the most efficient in removing free water from beneath the pavement structure and has been adopted by Illinois as the standard treatment for Interstate highways.

17. Key Words	18. Distribution Statement
CRC pavement, stabilized subbase, granular subbase, subsurface drainage, deflection, cracking, strain, water observation wells	No restriction. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	40	

Form DOT F 1700.7 (8-69)

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INTRODUCTION

In order to maintain uniform support under continuously reinforced concrete (CRC) pavements, and thus insure satisfactory service performance, the need for a nonerodible subbase has long been recognized. This study was undertaken to determine significant relationships between subbase type and CRC pavement behavior, to expand knowledge on stress levels and ranges in the steel and concrete of CRC pavement, and to evaluate three alternate methods of shoulder drainage. To achieve these objectives, an experimental project having three types of stabilized subbase, lime stabilized soil as subbase, and three alternate methods of shoulder drainage, was constructed southwest of Springfield, Illinois in 1971.

In 1961, the Illinois Department of Transportation began an intensive study of CRC pavements in cooperation with the Federal Highway Administration. The study included construction of several experimental sections of CRC pavements throughout the State to determine the significant relationship that exists between pavement behavior and certain design variables. One project was carefully instrumented for intensive study. The remaining projects were constructed as observation pavements involving only routine observations, measurements, and tests. The instrumented pavement was constructed in 1966. Because of the relatively short length of pavement usable for research, that part of the experimental work which was to include subbase type and alternate methods of shoulder drainage as variables was included in an adjacent section, Project F-277(15), FA 196, Section 1, Sangamon County.

This report is one of a series of reports (1, 2, 3, 4, & 5)[1/] prepared under the study, and covers the research phase relating to subbase type and drainage. It describes the design variables, construction and instrumentation of the experimental features, discusses the observations and measurements programs, and presents the conclusions that have been drawn.

Construction of the experimental subbase began on May 17, 1971. The slab was constructed between July 16 and August 11, 1971. It was opened to traffic immediately upon completion. This construction section of CRC pavement is a little more than 3 miles (4.8 Km) long, but the length of pavement suitable for research is about one mile. The mainline pavement in the test area is an 8-inch-thick continuously reinforced portland cement concrete slab. The longitudinal reinforcement consists of deformed bars meeting the requirement of ASTM designation A 615, Grade 60, for billet-steel bars. Transverse reinforcement consisted of deformed bars meeting ASTM designation A 615, Grade 40 billet-steel, chosen to permit welding of support chairs to the transverse bars. The experimental pavements were designed with the longitudinal steel amounting to 0.6 percent of the cross-sectional area of the pavement. The longitudinal steel consisted of No. 5 bars at 6½-inch (165-mm) centers. The transverse reinforcement was No. 3 bars at 25-inch (635-mm) centers. The reinforcement bars were assembled as a continuous mat on the subgrade by means of chairs extending upward to the mid-depth of the slab. Splice laps were maintained 16 inches (406 mm) in length. Concrete was deposited to full pavement thickness in a single operation and finished by a slipform paver.

1/ Numerals in parentheses refer to references listed at conclusion of report.

DESCRIPTION OF PROJECT

The experimental project is located southwest of Springfield on Route US 36 (Springfield Bypass). The annual precipitation averages about 35 inches (889 mm). The temperature ranges from about 77°F (25°C) in the summer to 29°F (-1.7°C) in the winter, with an annual average of 53°F (11.7°C). Normally, the frost penetration averages about 21 inches (533 mm).

The main soils on the site of the experimental project are A-6, but soils of types A-4 and A-7 also exist.

This research project was treated as a complete factorial design, including one slab thickness, 8 inches (203 mm); one reinforcement type (deformed reinforcement bars); one depth of reinforcement (mid-depth in the slab); and four subbase types, 8-inch (203-mm) lime-stabilized soil mixture (LSSM), 4-inch (102-mm) granular material, 4-inch (102 mm) bituminous-aggregate mixture (BAM), and a 4-inch (102-mm) cement-aggregate mixture (CAM). There are four main test sections and three replicate sections. Three test sections and three replicate sections are located in Construction Section 1. One test section of 8-inch (203-mm) pavement on granular subbase was "borrowed" from the adjoining construction Section 2-1 constructed five years before, and incorporated into this study. Various details of the test sections, including the underdrain sections, are given in Figure 1.

In addition to the main experimental features, two different designs of longitudinal underdrains were installed in the westbound roadway of the BAM subbase section to compare the effects of the underdrains on behavior of the pavement and paved shoulders with those of the standard treatment of open-graded subbase beneath the paved shoulder daylighted to the side slope. The eastbound

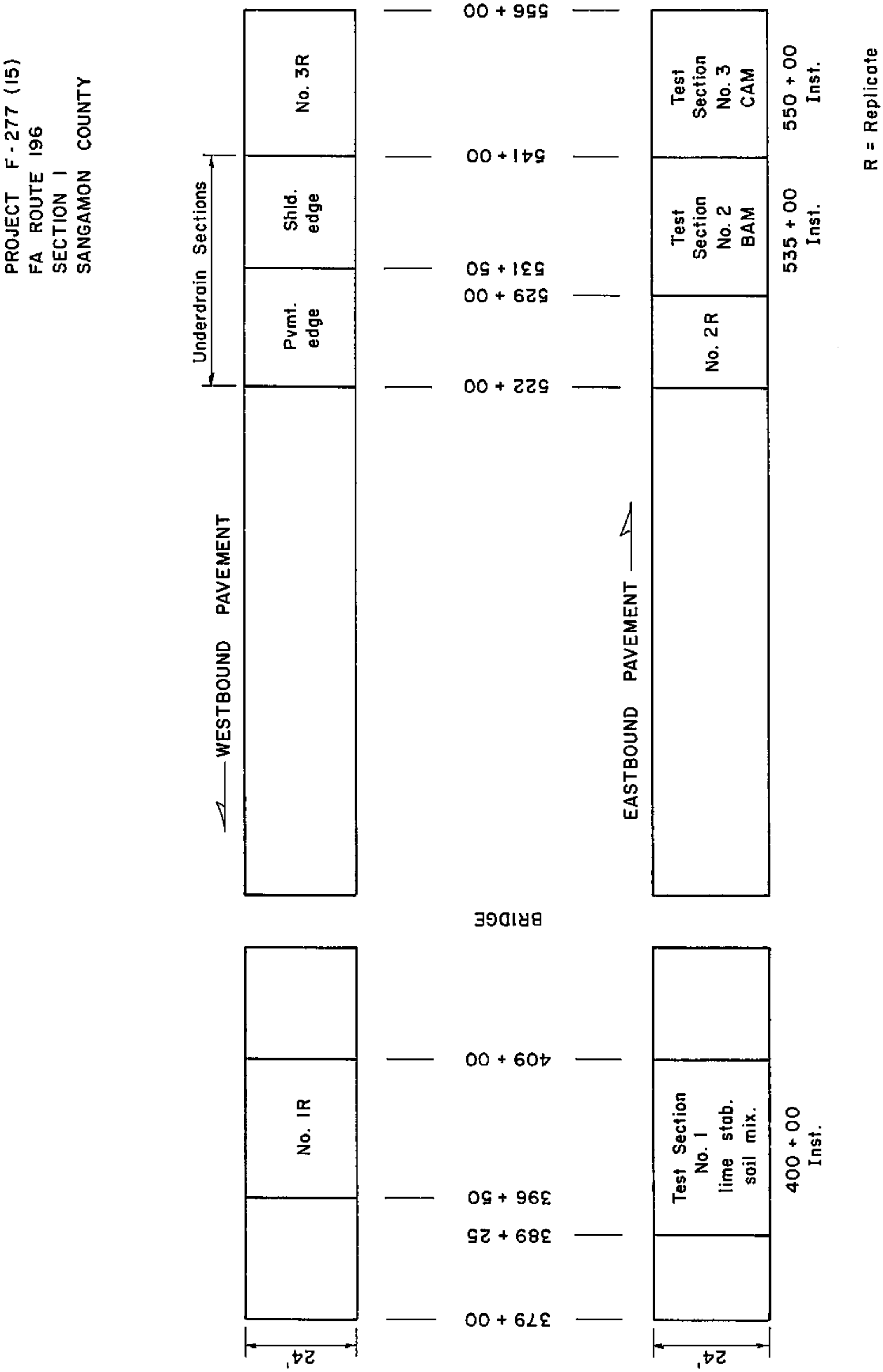


Figure 1. Location Of Test Sections.

pavement opposite the experimental underdrains was used as the control section for the standard treatment. Various details of the two underdrain test sections and one control section are given in Figure 2.

CONSTRUCTION OF EXPERIMENTAL FEATURES

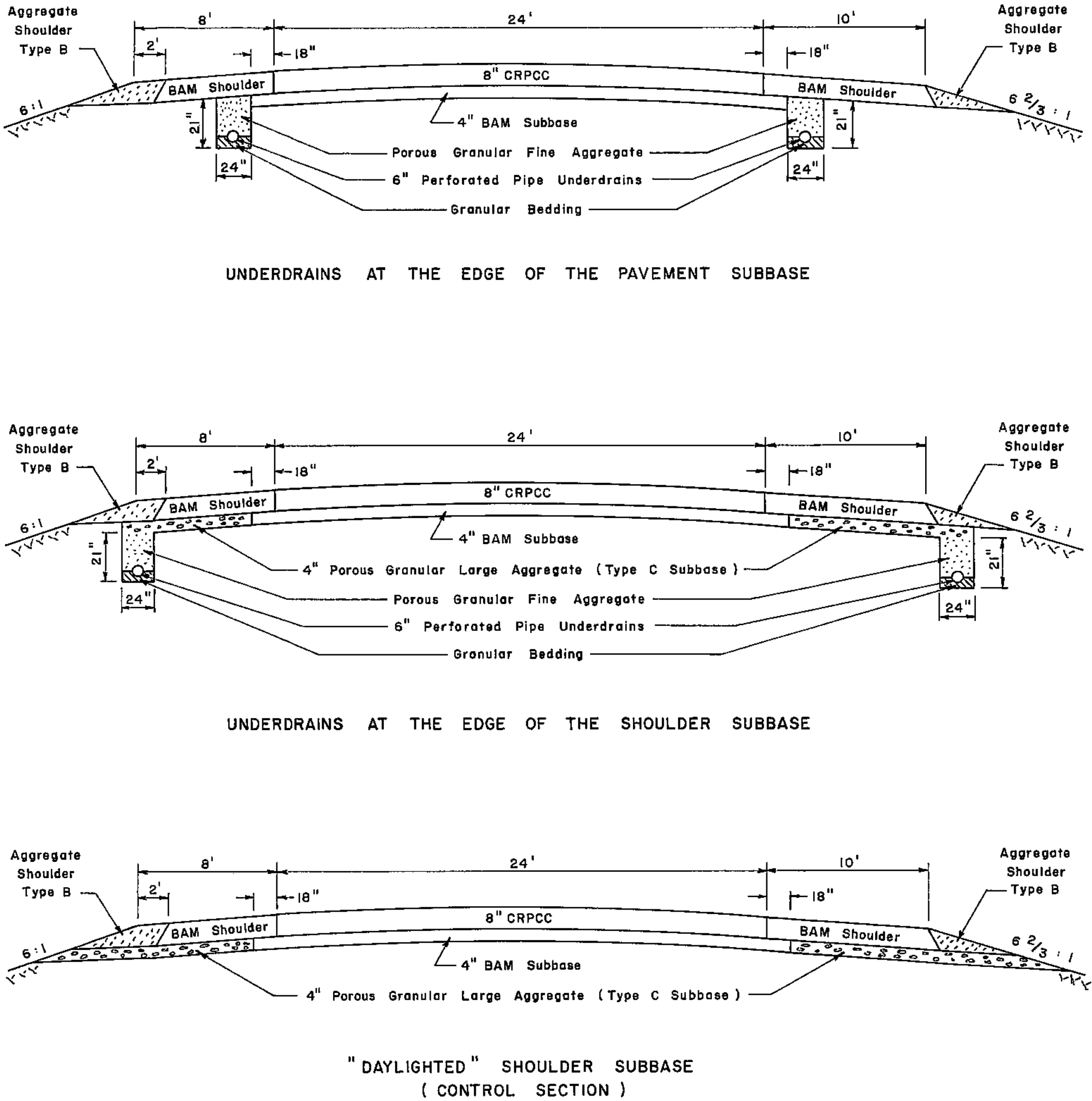
Cement-Aggregate Mixture (CAM)

The cement content of the CAM mixture was set at 8 percent of the dry weight of the aggregate. The aggregate was a blend of 75 percent limestone coarse aggregate and 25 percent natural sand. Specification limits for the gradation of the aggregate blend were as follows:

Sieve Size	Specification Limits (total passing percent by weight)
1 inch*	100
½ inch	60 - 100
No. 4	55 - 75
No. 8	40 - 65
No. 200	5 - 12

* 1 inch = 25.4 mm

The constituents of the mixture were accurately proportioned and thoroughly mixed in a mechanical mixer at a central mixing plant and trucked to the jobsite. The placement of CAM for the full width of the subbase was done with a CMI Autograder which incorporated automatic control of both its vertical and horizontal movement. The vertical controls of the CMI Autograder were set slightly greater than the 4-inch (102-mm) required thickness. The CAM subbase was deposited, spread, compacted to the required density, and shaped within two hours after the time the water was added to the mixture. Shaping of the surface took place near the completion of compaction. The density of the compacted CAM subbase was determined by a nuclear densometer. The control limits and the average of the test results were as follows:



Note : 1 inch = 25.4mm
1 foot = 0.305m

Figure 2. Details Of The Two Underdrain Test Sections And One Control Section.

DRY DENSITY				MOISTURE	
Standard Proctor, lb/cu ft	Actual Compacted, lb/cu ft	Percent of Standard	Spec. Reqd. (%)	Optimum (%)	Actual Content* (%)
143.5	136.9	95.5	94	6.9	5.8

*Average of 13 tests
1 lb/cu ft = 16/03 Kg/m^3

Following completion of the CAM, a curing coat of asphalt RC-2 was applied to the surface. The CAM was placed on May 17 and 18, 1971. The highest temperature was 85°F (29.4°C) and the lowest was 60°F (15.6°C).

Two cores were taken from the completed CAM subbase at the age of 18 days to determine the compressive strength. One was taken at the centerline of the westbound mainline at Station 546+20 and the compressive strength was 2120 psi (14,616.9 kPa). The second core was taken at the centerline of the eastbound mainline at Station 543+50 and its compressive strength was 2370 psi (16,340.6 kPa). These cores were soaked in water for three days prior to testing.

Bituminous-Aggregate Mixture (BAM)

The BAM subbase consisted of a crushed stone coarse aggregate and a 100-120 penetration grade paving asphalt set at 4.6 percent by weight of the total mixture. The specified gradation specification limits for the aggregate and the typical aggregate gradation were as follows:

Sieve Size	Percent passing by weight: Spec. Range	Percent passing by weight: Average
1½ inch*	100	100
1 inch	90 - 100	90.5
½ inch	60 - 90	62.9
No. 4	35 - 55	41.9
No. 16	10 - 40	24.1
No. 200	4 - 12	5.5

* 1 inch = 25.4 mm

The results of tests on a typical sample taken at the jobsite were:

Marshall Stability (lb)	= 2940 lbs
Flow (0.01 inch)	= 8.3
Asphalt content (percent)	= 4.9

Note: 1 inch = 25.4 mm
1 lb = 0.454 kg

The BAM was placed for the full width and thickness with a CMI Autograder, and compacted immediately after spreading. The density of the finished BAM subbase was determined from cores taken from the completed work. The specifications called for the BAM subbase to be compacted at least 90 percent of theoretical density. The average density from six cores taken from the completed subbase was 95 percent.

The major portion of the BAM in the research area was placed on June 28 and 29, 1971. The highest temperature was 96°F (35.6°C), and the lowest temperature was in the range of 65°F to 68°F (18.3°C to 20.0°C).

Lime-Stabilized Soil Mixture (LSSM)

The soil placed in the top 12 inches (305 mm) of embankment within the area of the lime-stabilized soil subbase was a select soil classified as A-6(11) (AASHTO Classification) that had previously been determined by laboratory tests to be reactive with lime, and satisfactory for use in this work. A requirement for the selection of the soil for the mixture was that the compressive strength of the mixture should be at least 50 psi (345 kPa) greater than that of the non-treated soil. The lime content was set at 3.5 percent by weight of dry soil. The average compressive strength was determined to be 239 psi (1647.7 kPa) for the mixture, and 69 psi (476 kPa) for the untreated soil.

The subgrade soils to be stabilized were scarified and dry lime was spread over the surface by a screw-type spreader. Water was added during the initial mixing operation in a sufficient quantity to bring the moisture content at least three percent above the optimum. After initial mixing, the mixture was sealed by rolling and then left to condition for a period of at least 48 hours. After the conditioning period, the final mixing was accomplished by a CMI Autograder. The mixture was then compacted to the required density and trimmed to final grade by the CMI Autograder.

It became apparent after the final finishing that proper stabilization of the soil had not been accomplished. The loose condition of the surface and the rutting of the surface that resulted from the truck used for the curing operation indicated instability of the mixture. Checks on the depth of the stabilization showed that instead of the required 8 inches (203 mm), an average of 5 inches (127 mm) was obtained in the eastbound lane and 3.75 inches (95 mm) in the westbound lanes. A decision was made to restabilize the middle 27 feet (8 m) with an additional 3.5 percent lime and to extend the depth to 12 inches (305 mm). The middle 27 feet of the roadway embankment was plowed to a depth of 12 inches (305 mm) by a farm plow, disked to remove the clods, and the lime and water were added and the mixture reprocessed, compacted, and trimmed as before.

The standard dry density and optimum moisture content of the LSSM were determined to be 98.2 lbs per cu ft (1574 Kg/m^3) and 22.1 percent in accordance with AASHTO Designation T99 (Method C). The mixture was compacted to an average density of 99.1 percent of the standard at an average moisture content of 20.6 percent.

The lime stabilization was carried out between June 24 and July 8, 1971. The highest temperature was in the upper 80's to lower 90's, and the lowest temperature was in the upper 50's to lower 60's. Most of the time the weather was warm and dry.

Granular Materials

As previously mentioned, the test section containing the granular subbase was borrowed from the adjoining construction section and had been completed in 1967. The material was a crushed limestone coarse aggregate conforming to the following:

Sieve Size	Specification Limits (percent passing)	Typical Gradation (percent passing)
1 inch	100	100
½ inch	60 - 90	90
No. 4	40 - 60	56
No. 8	25 - 50	37
No. 16	20 - 40	25
No. 200	5 - 15	12

Underdrains

The two designs of longitudinal underdrains were installed in the westbound roadway of the BAM subbase section between Stations 522+00 and 541+00. The eastbound pavement opposite the experimental underdrains was used as the control section and included the daylighted granular subbase beneath the paved shoulder.

The first underdrain system (Station 522+00 to 531+50) included a 6-inch diameter (153-mm) perforated pipe placed parallel to and 18 inches (457 mm) from each edge of the pavement. In this system, the BAM shoulder was placed directly on the earth subgrade.

In the second system (Station 531+00 to 541+00), a 4-inch (102-mm) thick, Type "C" granular subbase (open-graded coarse aggregate) was placed for the full widths of the paved shoulders and the 6-inch (152-mm) diameter perforated pipe was placed along the outer edges of the shoulders.

In both underdrain systems, the perforated pipes were placed approximately 2 feet (0.60 m) deep in a trench backfilled with porous granular material (sand). Outlet drains of non-perforated pipe were placed at right angles to the pavement at approximately 200-foot (61-m) intervals along the perforated pipe outletting into the median and side ditches. The underdrains were constructed prior to the construction of the shoulder and Type "C" subbase but after construction of the mainline subbase and pavement.

The standard treatment Type "C" granular subbase (open-graded coarse aggregate) was constructed under the paved shoulders to conduct the influx water along the shoulder subgrade through the subbase to the side slopes within the limits of the control section. In this system no underdrain pipes were used. The thickness of the "daylighted" layer was 4 inches (102 mm), and the BAM shoulder pavement was placed directly on this layer. This control section is located between Stations 522+00 and 541+00 on the eastbound roadway.

INSTRUMENTATION INSTALLATION

Instruments for measuring strain and temperature were installed during pavement construction at three locations in the eastbound pavement. They were designed to determine the strain in the longitudinal reinforcing steel and concrete in the vicinity of induced cracks and uncracked pavement. In addition, observation well points were installed in the outer shoulder at three locations in each drainage test section to measure the effectiveness of the different designs in removing free water from beneath the pavement and shoulders.

Instrumented Panels

One instrumented panel was installed near the center of each of the main test sections. Locations of the instrumented panels are Stations 400+00 EB, 535+00 EB, and 550+00 EB. All instruments in the panel were located within the length of one reinforcing bar in the driving lane of the pavement. All instrumented panels were similar in arrangement and included strain gages, thermocouples, concrete strainometer, and brass plugs for Whittemore gage.

To insure the development of a crack at a predetermined location in relation to the installed instruments, a crack was induced by placing a 20-gage galvanized steel strip 2 inches (51 mm) high and about 24 feet (7.3 m) long on the subbase under the longitudinal reinforcement. This strip, which was fabricated to conform to the crown of the pavement, was securely held in a vertical position by anchoring it to the subbase during the placing and finishing of the concrete.

It was hoped that adjacent cracks would be far enough away from the induced crack that the effect of the induced crack on the strains in the steel and concrete could be isolated. Unfortunately, additional cracks developed within the instrumented panels at all locations.

The strain gages which were installed were Bakelite Back SR-4 electrical resistance foil gages. Each gage had a gage factor of 2.0 to 2.1, and gage resistance of 120 ohms. The nominal length of the gage was 5/8 inch (15.9 mm), with a trimmed width of 1/4 inch (6.4 mm). There were 18 SR-4 strain gages per instrumented panel and each gage was located as shown in Figure 3. Strain gages numbered 1 through 12 were located on the longitudinal reinforcement. Strain gages numbered 13 and 14 were designed to measure temperature effects. These gages were mounted on opposite sides of a short piece of reinforcing steel, and were embedded at mid-depth of the pavement.

Two pairs of gages (Gage Nos. 15 and 16, and 19 and 20) were bonded to Vycor (silica glass) rods, which were placed in styrofoam-insulated boxes and embedded in each instrumented panel. The Vycor material has a very low coefficient of thermal expansion, and the gages bonded to this material should not

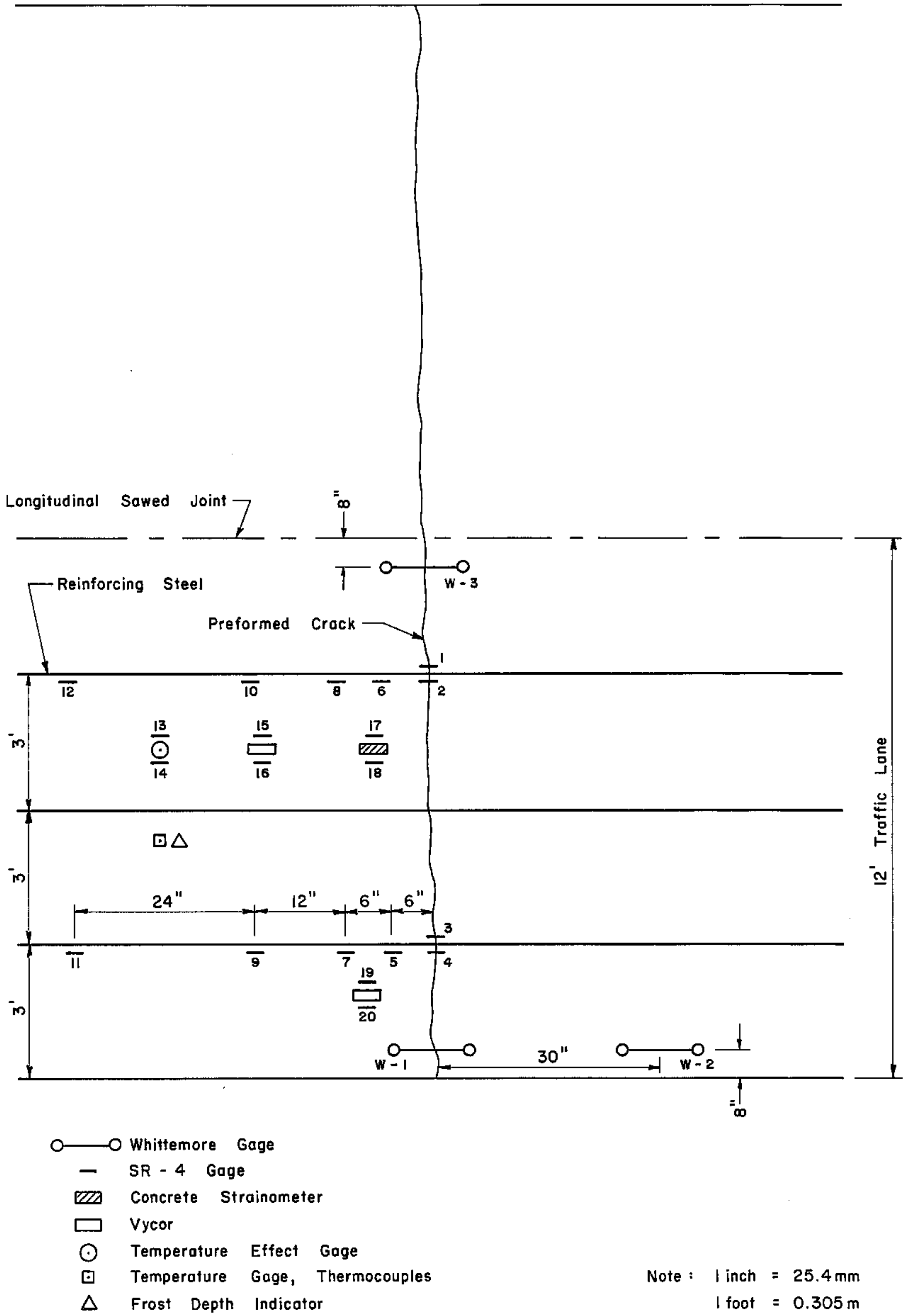


Figure 3. Location Of Strain Gages.

be affected much by a change in temperature. These gages were to serve as a zero reference to check the long-term drift of the instrumentation.

The gages attached to Vycor were wired in such a way that three complete Wheatstone bridges could be formed from the two Vycor assemblies. Unfortunately, the strain readings from half of the Vycor-mounted gages indicated fluctuations of more than ± 500 micro-inches per inch. There was no trend to show that the strain variations were directly related to changes in temperature. Every change in these gages also affects the steel strain data because the Vycor-mounted gages were used as adjacent arms of a four-arm bridge circuit when readings were made.

An attempt was made to determine the strain in the concrete itself by installing a concrete strainometer. Gage Nos. 17 and 18 are installed on the shop-made concrete strainometer body. The concrete strainometer was embedded near the middle of the instrumented panel.

The procedure, as suggested by the manufacturer, for installing strain gages consisted of removing deformations from the reinforcing steel, applying epoxy to bond the SR-4 gage to the steel, attaching lead wires, and waterproofing the installation.

Thermocouples were installed to record changes in the internal temperature of the slab. Thermocouples T-1 through T-13 were located ± 4 feet (1.22 m) from the induced crack and near the middle of each instrumented mat. Gage No. T-1 was 1 inch (25.4 mm) below the pavement surface, No. T-2 was at reinforcing steel level, and No. T-3 was 1 inch (25.4 mm) above the bottom of the concrete. Gage No. T-4 was located at the middle of subbase, and Gage Nos. T-5 through T-13 were spaced at 4-inch (101.6-mm) intervals in the embankment.

Three sets of Whittemore gage plugs per instrumented panel were installed to determine the surface width of the induced crack. Two sets of plugs were at the induced crack, and one set was 2 1/2 feet (.76 m) from the induced crack.

At each location, the leads from the gages were extended under the pavement to the outer edge, and then extended in a common trench 9 to 12 inches (228.6 mm to 304.8 mm) below the earth subgrade across the shoulder area and 2 to 4 feet (.61 m to 1.22 m) down the side slope.

Despite the care taken in installing the strain gages and the steps taken to try to obtain reliable readings over a long period of time, the formation of random cracks within the instrumented panels in addition to the induced cracks, and the fluctuations in the Vycor-mounted gages, created more variables and made it impossible to evaluate the steel strains at the induced cracks and at the intended intervals away from the cracks. Consequently, no strain data are presented in this report. The crack patterns which developed and the dates of crack formation in the CRC slab on CAM, PAM, AND LSSM subbase are shown in Figures 4, 5, and 6, respectively. A similar plot for the section on granular subbase borrowed from the previously constructed adjacent construction project is shown in Figure 7.

Water level observation wells were installed in each of three experimental drainage systems. The wells were installed 2, 5, and 9 feet (0.61, 1.5 and 2.1 m) from the pavement edge in the outer shoulder at three locations in each drainage design subsection. The observation wells consisted of steel tubing 7/8 inch O.D. (22.3 mm), 14 inches (356 mm) long, drawn to a point at the lower end, and perforated for a distance of 4 inches (101.6 mm) above the point. The tubes were forced through drilled holes in the shoulder pavement in the embankment

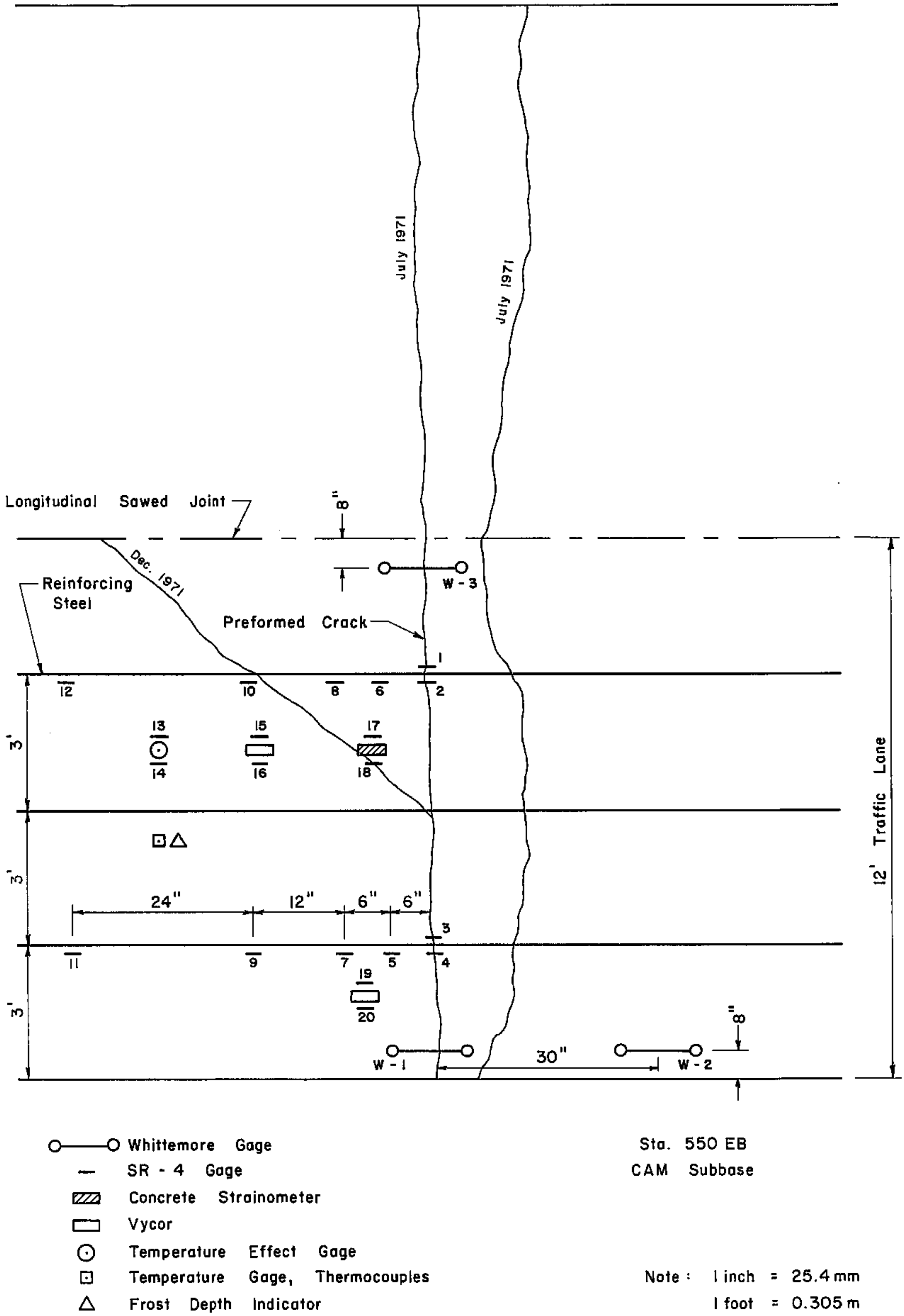


Figure 4. Crack Pattern For CRC Slab On CAM Subbase.

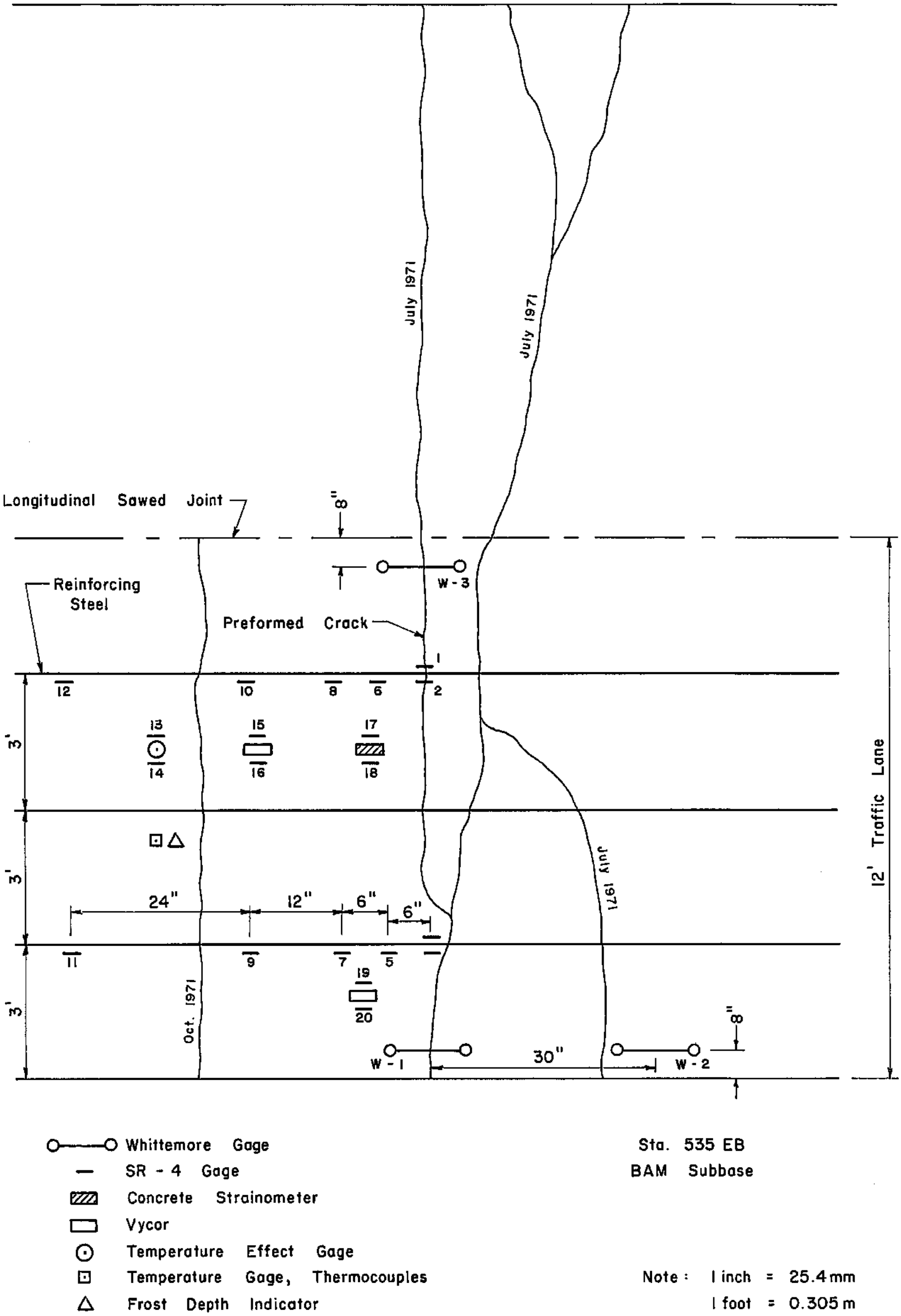


Figure 5. Crack Pattern For CRC Slab On BAM Subbase.

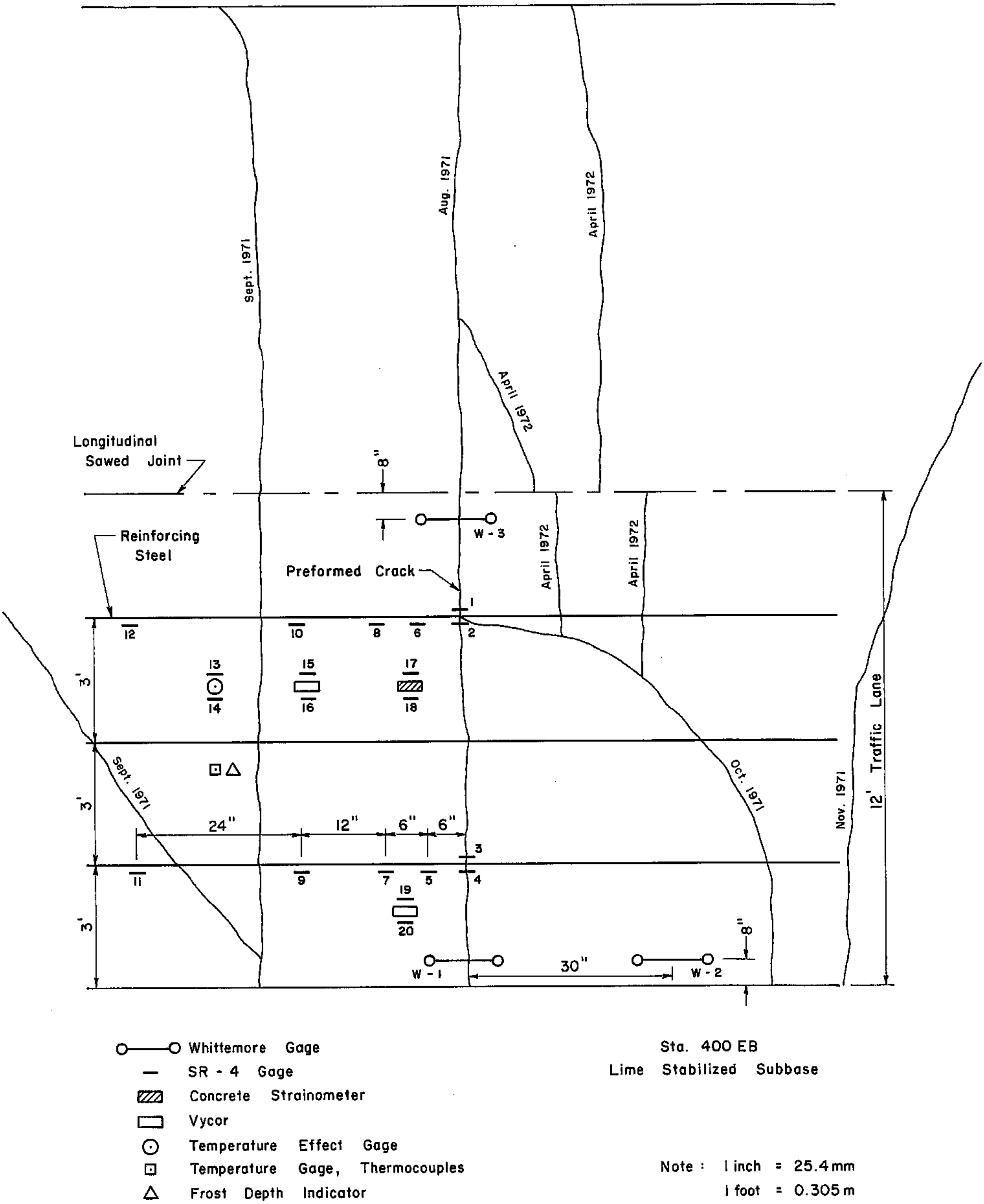


Figure 6. Crack Pattern For CRC Slab On Lime Stabilized Subbase.

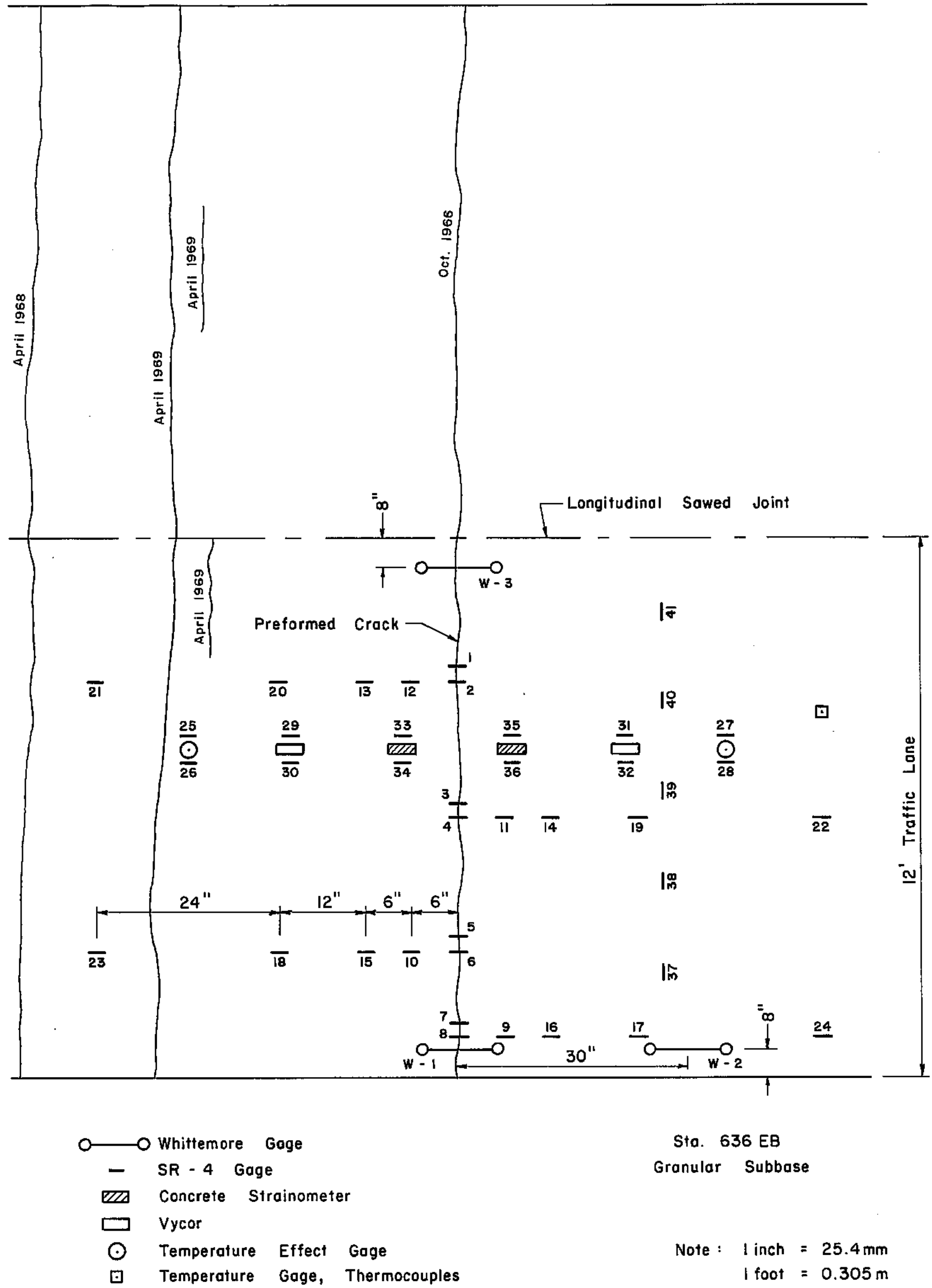


Figure 7. Crack Pattern For CRC Slab On Granular Subbase.

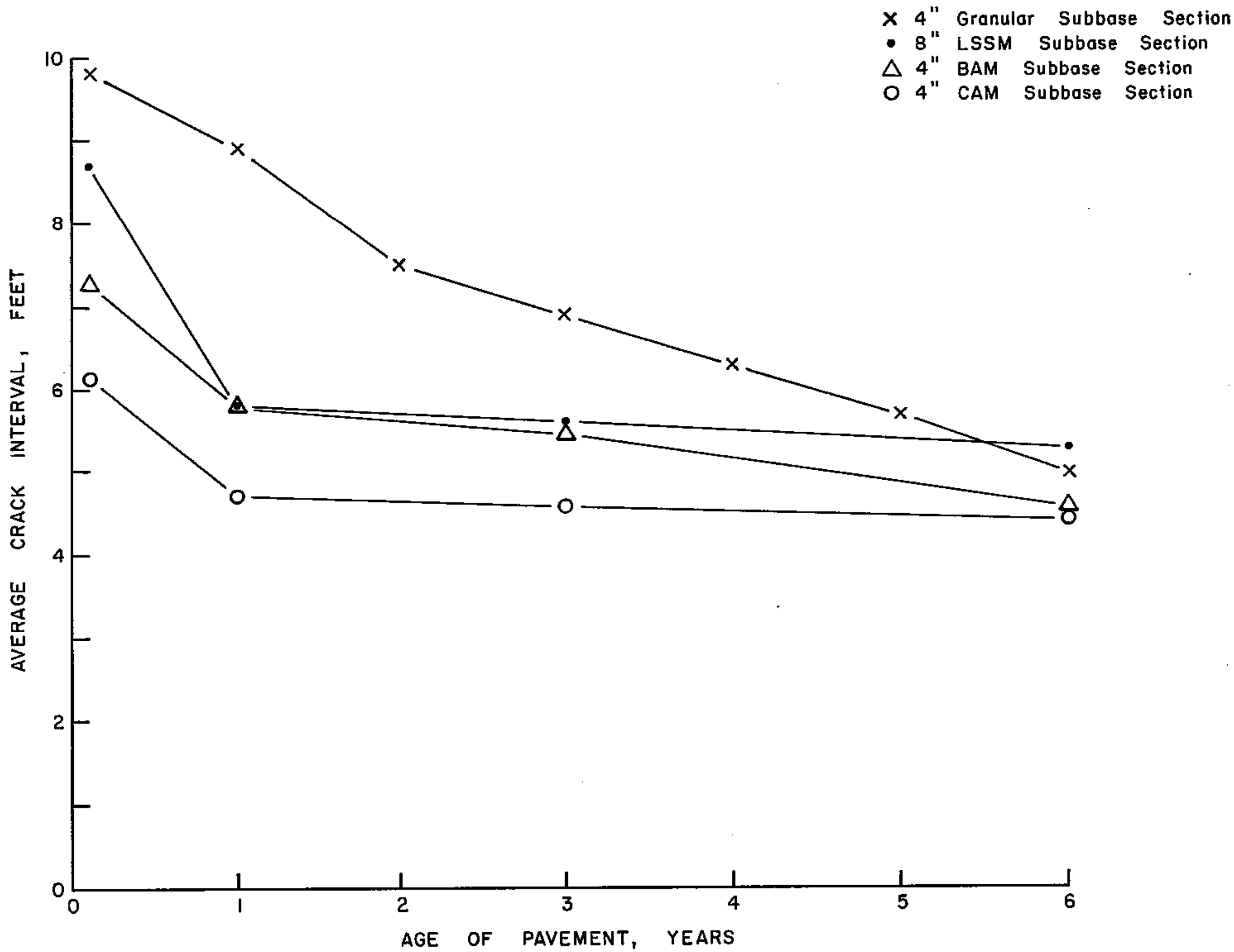
soil until the upper end was 1/4 inch (6.4 mm) below the shoulder surface. When not in use, the tubes were plugged with expandable rubber plugs to prevent influx of surface water. A total of 27 wells were installed.

TRANSVERSE CRACKING STUDY

Upon completion of construction of this experimental project, an intensive crack survey was made of all the main and replicate sections in an attempt to determine if subbase type has an effect on transverse cracking of CRC slabs. These surveys were repeated generally every two years. The average crack interval representing each crack survey was calculated and is plotted in Figure 8. It is seen that for the slabs on CAM, BAM, and LSSM most of the transverse cracking developed during the first year following construction, while transverse cracking in the granular subbase sections developed at a slower rate and continued to develop over the six-year measurement period. At the end of the six-year period, the differences in average crack interval among the various subbase-type sections were relatively small.

The average crack interval ranged from 4.7 feet to 8.9 feet (1.43 m to 2.7 m) at the end of the first year and from 4.5 feet to 5.3 feet (1.4 m to 1.6 m) at the end of 6 years. At the end of 6 years, the average crack interval for all test sections is within the desired range of 3 feet to 10 feet (0.9 m to 3.0 m). It does not appear that subbase type had much influence on the number of transverse cracks that developed in the slabs.

The latest condition survey data (at age six years) were further processed to assess the effect of the subbase type on the uniformity of the transverse crack pattern. The cracks were grouped into cells, depending on the interval between individual cracks and plotted as bar graphs. Also, the number of



Note : 1 foot = 0.305 m

Figure 8. Relationship Between Average Crack Interval And Pavement Age.

converging or "Y'ing" cracks were counted and reduced to number of cracks per 100 feet of pavement. This type of cracking occurred intermittently along the length of pavement in each test section, usually in areas of closely spaced cracks. The results are shown in Figure 9.

As can be seen in Figure 9, the crack pattern in the slab constructed on LSSM appeared to be the most uniform, with 68 percent within the desired range of 3 to 10 feet. The average crack interval was greatest at 5.3 feet, and the number of undesirable "Y'ing" cracks was least at 3.8 per 100 feet of pavement. The slab constructed on BAM subbase section was second, with 56 percent within the desired range; the slab on CAM subbase was third, with 52 percent within the desired range, and the granular subbase section was lowest with only 45 percent within the desired range. This test section developed the greatest number of cracks spaced both more than 10 feet (3.1 m) apart and less than 3 feet (0.9 m) apart, but developed less "Y'ing" cracks than any except the LSSM section. The granular subbase is the only test section which is a "trenched"-type design. This type of subbase design has a tendency to collect free water and retain it for extended periods of time (5), which may have affected the cracking uniformity, especially after the pavement had passed through six yearly cycles.

It was observed that the performance of the pavement at transverse cracks in all test sections after six years was excellent. The cracks appeared to be tight at the pavement surface, and had a fairly uniform pattern. No distressed areas or cracks were found.

DEFLECTION STUDY

A static rebound edge deflection study was carried out in an attempt to assess the influence, if any, of subbase type on the pavement's structural

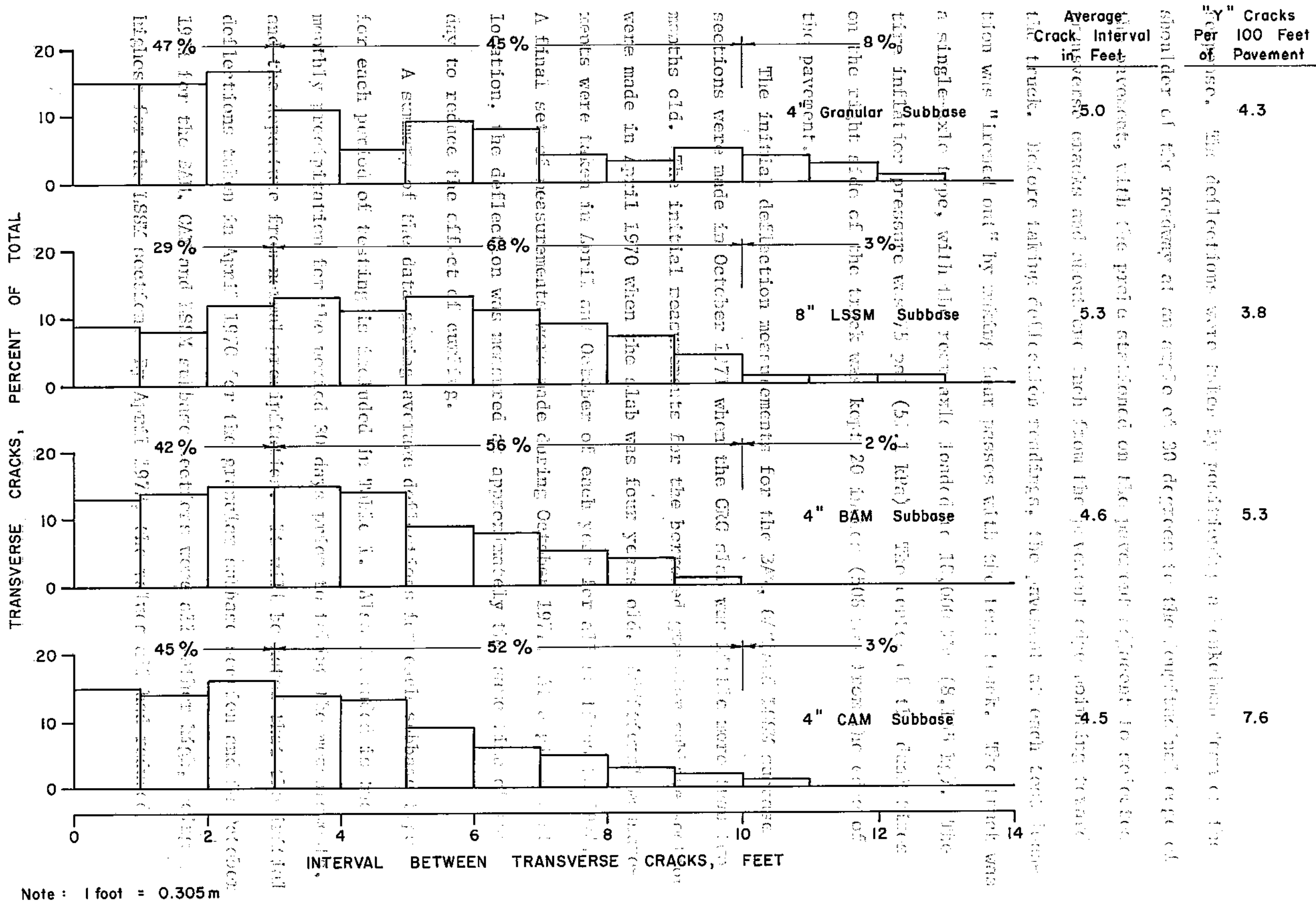


Figure 9. Information Relative To "Y" Cracks And Average Crack Interval.

TABLE 1

SUMMARY OF STATIC REBOUND DEFLECTION MEASUREMENTS BY SUBBASE TYPE

Date of Testing	Precipitation & Departure from Normal 1/ - (in.)*		Average Static Rebound Deflection - (in.)*			
	Precip.	Departure	LSSM Subbase	BAM Subbase	CAM Subbase	Granular Subbase
April 1970	7.73	+4.80				.020
October 1971	3.76	+0.83	.027	.020	.021	.012
April 1972	4.03	+1.15	.011	.015	.012	.013
October 1972	3.95	+1.02	.015	.015	.014	.014
April 1973	7.89	+5.01	.013	.014	.015	.018
October 1973	3.28	+0.35	.012	.013	.012	.013
April 1974	3.39	+0.69	.012	.012	.010	.016
October 1977			.010	.012	.011	.010

1/ For the period commencing 30 days prior to deflection measurements

* Note: 1 inch = 25.4 mm

appreciably and were more in line with values obtained on other CRC pavement of similar design in Illinois. The reason for the initial high deflections is not known.

No relationship between the magnitude of deflection and type of subbase is apparent from the analysis of the deflection data. As can be seen in Table 1, the average deflection of each test section fluctuated from Spring to Fall, but there is no significant trend. Also, the fluctuations do not appear to correlate with the amount of precipitation preceding the testing. The difference in average deflections for the final set of measurements was only 0.002 inch (0.5 mm) for the various subbase types, a value not believed to suggest any significant difference in support due to subbase type. The age of pavements at this time was six years for the BAM, CAM, and LSSM subbase sections, and 11 years for granular subbase section. As previously mentioned, the granular subbase section was the only one that was of a trench-type design. It should be noted that longitudinal underdrains were added along the outer edge of the pavement in the fall of 1972.

In conjunction with static deflection measurements, crack width measurements were taken with the load on and off the induced cracks. The crack width measurements were made with a Whittemore strain gage and reference plugs set in the pavement surface on each side of the induced cracks. These induced cracks, as discussed earlier, were preformed by inserting a 2-inch (50.8-mm) high sheet metal strip from the bottom of the slab. Therefore, the effective aggregate interlock thickness at these cracks is 25 percent lower than a normally developed crack. As expected, the crack width at the pavement surface decreased when the load was applied, and the crack width at the pavement surface decreased with increased pavement deflection. No trend which can be associated with subbase type was found.

SUBSURFACE DRAINAGE STUDY

During the 6½-year period following construction and opening of the experimental section to traffic, a considerable amount of subsurface drainage data have been obtained. The general condition of the shoulders and drainage systems has been observed. Detailed condition surveys of the CRC pavement within each drainage system have been made, and edge deflections were taken in test sections.

The water level in the observation wells, or the hydrostatic head, measured at frequent intervals following the influx of water into the drainage system, is taken as an indication of the efficiency of the system. The difference in the water levels between the three installed wells in one set is taken as an indication of the magnitude and direction of the hydraulic gradients along which the water is draining from the shoulder drainage system. The drop in the water level from one measurement date to the next indicates the rate at which water is leaving the system.

During the spring and summer of 1973, water levels in the observation wells were recorded regularly, at least monthly, and at 24-hour intervals following rainstorms totalling one inch or more precipitation, beginning on the morning following the storms. Four major rainstorms occurred during this period. The first occurred on April 22, with one inch of precipitation, the second on June 18, with 2.6 inches, the third on June 26, with 1.45 inch, and the fourth on July 21 through 23, with a total accumulation of 2.6 inches.

The results of the measurements showed that all three subsurface drainage systems were performing their intended function of removing free water from the pavement shoulder system. The systems with the positive longitudinal underdrains, and specifically the system with the underdrain placed at the edge of the stabilized subbase, performed the intended function much better and faster, however, than did the system which included the open-graded subbase beneath the pavement shoulder daylighted to the outside slope.

The data indicated that the drainage system with the longitudinal underdrain placed at the edge of the stabilized subbase was most effective in removing free water in the shortest time. The level of water remained below the level of the earth subgrade at all times in each of the four major rainstorms, and the water level in well No. 1 located over the tile dropped to the bottom of the well or below it by the third day following a rainstorm.

The subsurface drainage system with the longitudinal underdrain placed at the outer edge of the shoulder, and the open-graded Type C subbase granular material under the shoulder and extending to the pipe, appeared to be the second most efficient system in removing subsurface water. Water levels were at the level of the earth subgrade or below it within only a few hours after the heavy rain had stopped.

The subsurface drainage system, including the daylighted Type C granular subbase beneath the paved shoulder, also removed free water from the pavement system, but at a slower rate than the other two systems. A head of water built up in the open-graded granular subbase beneath the shoulder as the water percolated toward the outside shoulder slope. In two of the four rainstorms during which measurements were taken, the head of water that built up in the granular subbase beneath the paved shoulder had not been completely dissipated by the morning of the third day following the rainstorm.

It should be noted that the shoulders were relatively new at the time of the above-described observations and, that as time passes and the shoulders age and openings develop in the joint between pavement and shoulders, the systems will be required to handle more and more water while at the same time undergoing a loss in efficiency with time. This should be particularly true for the daylighted system due to infiltration of soil fines and vegetation growth along the exposed face of the sideslope.

The subsurface drainage system with the underdrain at the edge of the stabilized subbase, by nature of its design, contains the least amount of air space in the substructure to serve as a reservoir for free water. It should absorb less water than the other designs, shed more surface water as runoff, intercept water from the pavement area before it reaches the shoulder area, and keep the subgrade drier. Its main disadvantage over the other two systems is that it does not offer the added structural strength to the paved shoulder contributed by the Type C subbase.

The general condition of the shoulder surface in each drainage system frequently has been observed during the course of the study. In each test section the shoulders are performing eminently well after 6½ years of service. During the 1977 survey, it was observed that a few transverse cracks in the outside shoulders have developed. These transverse cracks did not appear to be associated to the type of subsurface drainage system. At some of these cracks, minor distress in the form of initial stages of alligator cracking has developed. This minor distress was limited to a very small area either side of the transverse crack at the pavement edge. It was observed during heavy rain that the excessive infiltration of the surface runoff at the pavement edge was travelling through some of the transverse cracks and was reappearing at the surface near the middle of the shoulders. During the freeze-thaw cycle this water may have been the main cause of the minor distress which was observed. No surface fatigue, consolidation or shear distress signs were observed during the 1977 survey.

Measurements from 1973 to 1977 were taken to evaluate the change in the shoulder surface relative to the pavement edge surface. These readings were taken in conjunction with the water level measurements. The measurements indicated that all shoulders have settled slightly relative to the pavement edge. The settlement was the smallest near the pavement edge, and it increased as the distance from the pavement edge increased. The least relative change has occurred in the shoulders on the daylighted subbase.

COST CONSIDERATIONS

No accurate information is available to the Division of Highways as to the Contractor's true costs for constructing the various stabilized subbases or underdrains for the experimental pavements. The only comparison available is based on the bid prices. The bid price for each type of stabilized subbase was as follows:

BAM - 4-inch (102-mm) = \$2.30 per sq yd

CAM - 4-inch (102-mm) = \$2.19 per sq yd

LSSM - 8-inch (203-mm) = \$1.43 per sq yd

As can be seen, the bid price for the CAM and BAM subbase was 53 percent and 61 percent, respectively, greater than the bid price for the LSSM subbase. Since the early seventies there has been a shortage of paving asphalt, and this shortage, combined with inflation, has increased the price considerably. The present (1978) State average bid prices for 4-inch (102-mm) CAM is \$3.80 per sq yd, for 4-inch (102-mm) BAM it is \$4.10 per sq yd, and for 8-inch (203-mm) LSSM is about \$2.25 per sq yd. Using these cost estimates, the price for CAM and BAM is 69 percent and 82 percent, respectively, greater than for the LSSM subbase. It would appear on the basis of construction cost, that the LSSM subbase is cheaper to construct.

The bid price for the control or "daylighted" subsurface drainage system was the lowest. In comparing the bid prices for the two systems with positive longitudinal underdrains, the system having underdrains at the edge of the outside edge of the shoulder and with Type C subbase beneath the shoulder cost almost 33 percent more than the system having underdrains at the edge of the pavement subbase.

SUMMARY OF PRINCIPAL FINDINGS

Observations and measurements of the behavior of the pavement constructed on the experimental subbases and the effects of the underdrains on the performance of the pavement and shoulders have indicated the following general trends:

(1) The type of subbase had little to no influence on the number of transverse cracks that developed in the pavement, as indicated by the crack survey data collected over a six-year period.

(2) Subbase type did have an effect on the uniformity of the crack pattern for transverse cracking in the CRC pavement slabs. Transverse cracks in the slab constructed over the lime-stabilized soil mixture as subbase were the most uniform, with 68 percent of the transverse cracks within the desired crack interval range of from 3 to 10 feet. The pavement slab on the BAM subbase was second with 56 percent. The CAM subbase section was third with 52 percent, and the slab constructed over the granular subbase was the lowest with only 45 percent within the desired range.

(3) No relationship between the magnitude of deflection and type of subbase was apparent from the deflection measurements taken on the experimental

pavements. The average deflection obtained from the various test sections fluctuated in magnitude from spring to fall, but indicated no specific correlation either with the season of the year or with the amount of precipitation preceding the testing.

(4) All three types of subsurface drainage systems were performing their intended function of removing free water from the pavement and shoulder structures. The two subsurface drainage systems with the positive longitudinal underdrains removed free water from the pavement section at a much faster rate than did the system which included the open-graded subbase beneath the pavement shoulder daylighted to the outside slope. The system with the longitudinal underdrains placed at the edge of the stabilized subbase appeared to be the most effective system in rapidly removing free water from the pavement shoulder structures.

(5) At the end of the 6½-year period during which the observations and measurements were taken, all pavement sections, including the various types of experimental subbase and subsurface drainage systems, were performing eminently well. All sections were completely free of any signs of distress, and exhibited no indications of any problems developing in the near future.

RECOMMENDATIONS AND IMPLEMENTATION OF FINDINGS

During the course of this study the findings have not indicated any significant difference in performance due to subbase type. This is contrary to past experiences in Illinois and other States wherein stabilized subbases have shown superior performance to unstabilized granular subbases. The lime-stabilized soil mixture as subbase, on the other hand, has performed as well as the other stabilized subbases. This method of construction offers a potential for reduced construction costs along with reducing the amounts of natural aggregate, portland cement, and asphalt required for new pavement subbase construction. Based on its excellent performance to date, it is recommended that additional experience be gained with the use of lime stabilization of the top of the earth subgrade and shoulder areas to function as subbase by constructing additional pavement sections with this type of subbase. Evaluation of the performance of the additional experimental sections will permit a proper evaluation of the relative merits of this type of construction over the now-standard stabilized subbase construction in Illinois.

At the time of the construction of these experimental pavement sections, Illinois had adopted a means of providing positive drainage of free water from beneath the pavement and shoulder structures. The standard in use at that time provided for the construction of either an open-graded Type C granular subbase beneath the pavement shoulder and daylighted to the outside slope or, as an alternate, a 6-inch longitudinal underdrain placed along the edge of the stabilized subbase. Practically all of the construction at that time included the Type C subbase rather than the longitudinal underdrains because of cost considerations. Later work indicated that the cost of the underdrains could be reduced by reducing the size from 6-inch diameter to a 4-inch diameter pipe, which would still have sufficient capacity for removing the free water. This fact, in combination with the fact that the experimental subsurface drainage sections on this project indicated that the efficiency of the longitudinal underdrain system placed along the edge of the stabilized subbase was more efficient in terms of removing the free water from the pavement shoulder structure at the fastest rate, resulted in the Department's revising its standards in 1975 to require the construction of 4-inch longitudinal underdrains for subsurface drainage along the edges of the stabilized subbase in lieu of other types of subsurface drainage systems.

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