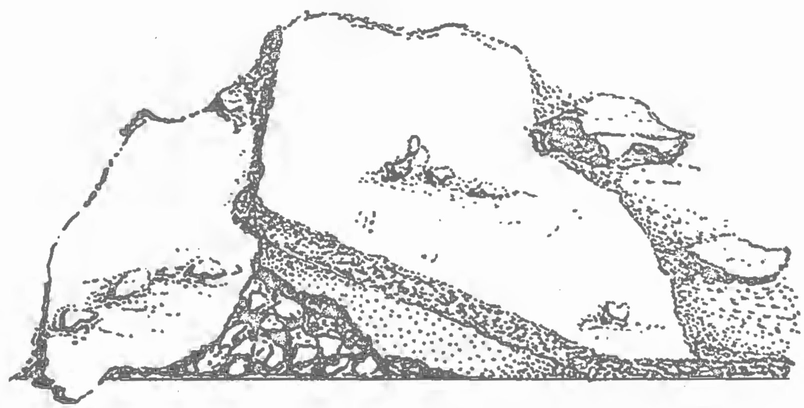
THE STUDY OF PAVEMENT BLOWUPS



ARKANSAS STATE HIGHWAY DEPARTMENT
PLANNING AND RESEARCH DIVISION
IN COOPERATION WITH
U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

RESEARCH PROJECT 10

THE STUDY OF PAVEMENT BLOWUPS

PREPARED BY
ARKANSAS STATE HIGHWAY DEPARTMENT
DIVISION OF PLANNING AND RESEARCH
IN COOPERATION WITH
U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

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FOREWORD

The purpose of this investigation is, first, to accurately determine the extent and magnitude of the pavement blowup problem in Arkansas; second, to indicate how the blowup causal factors have influenced the performance of Portland Cement concrete pavements; and, third, to show how the effects of these factors can be eliminated or minimized by design, construction, and maintenance practices.

AUTHORITY

Highway Research Project No. 10 (HRC-10), HPS-HPR-1(21), J456 was established June 6, 1963, by the approval of a detailed workplan under a joint agreement between the Arkansas State Highway Department, Planning and Research Division, and the U.S. Department of Commerce, Bureau of Public Roads.

PAVEMENT BLOWUPS

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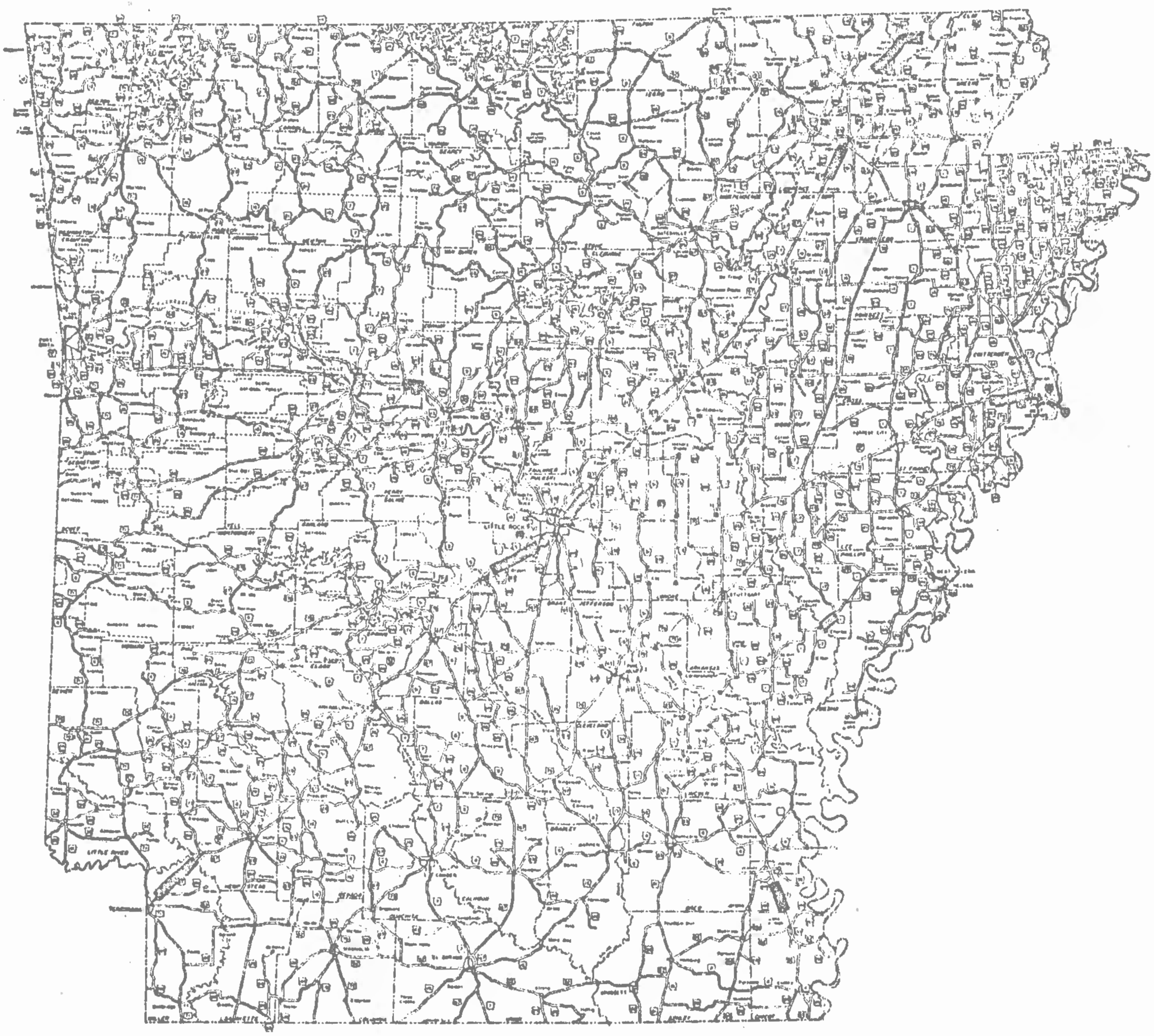
LOCATIONS OF STUDY



INTRODUCTION

This paper reports, in part, the results of the investigation of rigid pavement blowups. This project is being conducted by the Arkansas Highway Department in cooperation with the U.S. Bureau of Public Roads and is under the general supervision of the following Subcommittee of the Highway Research Committee:

M. S. Smith, Chairman

C. M. Matthews, Member

Pat Huddleston, Member

The inventory of 473 miles of rigid pavement revealed that 934 blowups occurred in 1963. This inventory was conducted by Maintenance Districts in each of the 10 Districts throughout the State. The pavement sections were then broken into three groups: (1) pavements constructed prior to 1944, (2) pavements constructed from 1944 to 1958, and (3) pavements constructed from 1958 to 1963. From the inventory sheets, one job from each group for each increment of 1.00 blowup per mile was selected as a study sample. Jobs under three miles long were eliminated unless they were needed for continuity of a section.

At the present, the study includes 202 miles of rigid pavement, which is approximately 35% of rigid pavement in service at the time the blowups were inventoried. Since the research started, the pavements have been regrouped into the classes of coarse aggregate which were used in the pavements. These groups were: (1) pavements constructed with natural washed gravel, and (2) pavements constructed with crushed stone.

THE HISTORY OF CONCRETE PAVEMENT

The current design of concrete pavements is based on long-proven or assumed fundamentals with regard to the effects of traffic, climate, subgrade conditions, and concrete properties on pavement thickness and jointing arrangements. Pavement designs are often modified through analytical equations, laboratory research, road tests, and pavement performance surveys. When any of the aforementioned studies are conducted, semi-empirical equations usually result.

The first concrete pavement built in the United States was in 1892 at Bellefontaine, Ohio. This pavement was built on the courthouse square and was designed for horse-drawn vehicles. The pavement texture had a grid-like pattern to assure safe footing for the horses. Some sections of this pavement are still in service.

The first section of concrete pavement in part for auto service was constructed in Wayne County, Michigan, in 1909. The engineers in that day, much like those of today, thought a road test might be in order before designing the pavement. The road test was called a "paving determinator". The testing apparatus consisted of a set of steel-shod shoes and a heavy iron-rimmed wheel mounted at opposite ends of a 20-foot pole and was designed to reproduce the traffic of horses and iron wagonwheels. The conclusion of that test was: "The concrete section laid under the specifications of the county commissioners of Wayne County, Michigan, showed by far the best resistance to the severe test to which pavements were put". The judgment of the Wayne County engineers and commissioners was substantiated by this test.

Another such concrete road for auto service was constructed in Jefferson County, Arkansas in 1911 and 1912; it was the historic Dollar-Way Road. The road, almost a 24-mile stretch was constructed by a Jefferson County Improvement District under the supervision of the State Highway Engineer. The design consisted of a nine-foot reinforced concrete roadway five inches thick with three-foot gravel shoulders. This road gave 18 years of service and some abandoned sections are still remaining.

In 1922 and 1923 the State of Illinois decided to get the motorists out of the mud. This decision got the famous Bates Test Road under construction. The test road consisted of 63 test sections of different designs, and World War I army trucks were used for establishment of traffic records. The test led engineers to the use of a longitudinal center joint to eliminate longitudinal cracking. The results were also used by Older to develop an equation relating pavement thickness to traffic loading, based on the theory of cantilevered beams.

In 1926 H. M. Westergaard of the University of Illinois developed equations for determining stresses and deflections in concrete pavements due to loads applied in the interior of the slab and at free edges and corners.

In the early 1930's, the Bureau of Public Roads conducted loading tests on concrete slabs at Arlington, Virginia. This test was to check Westergaard's theoretical equations. The Westergaard equations were modified to get closer agreement with measured stresses and deflections from the Arlington tests. Kelly, Spangler, and Pickett used the results of the Arlington tests to modify the Westergaard equations.

Since 1946, PCA has used these equations for design recommendations. PCA is now preparing a revised method of design based on transverse edge loading (1964).

The latest road test, the AASHO Road Test, was to develop additional equations and to study special problems by use of controlled test sections. The AASHO staff requested that a certain number of these sections be designed for failure. This was done and the designs were incorporated in the Test. An equation was developed by this road test for the design of rigid pavements. As a part of the conclusions reached, it is noted that insufficient concrete pavement failures occurred for thorough analysis.

From these tests and conclusions one might ask, what has been the progress since 1892? It can only be said that more research is needed to design a more trouble-free concrete highway.

CHAPTER I
GENERAL INFORMATION

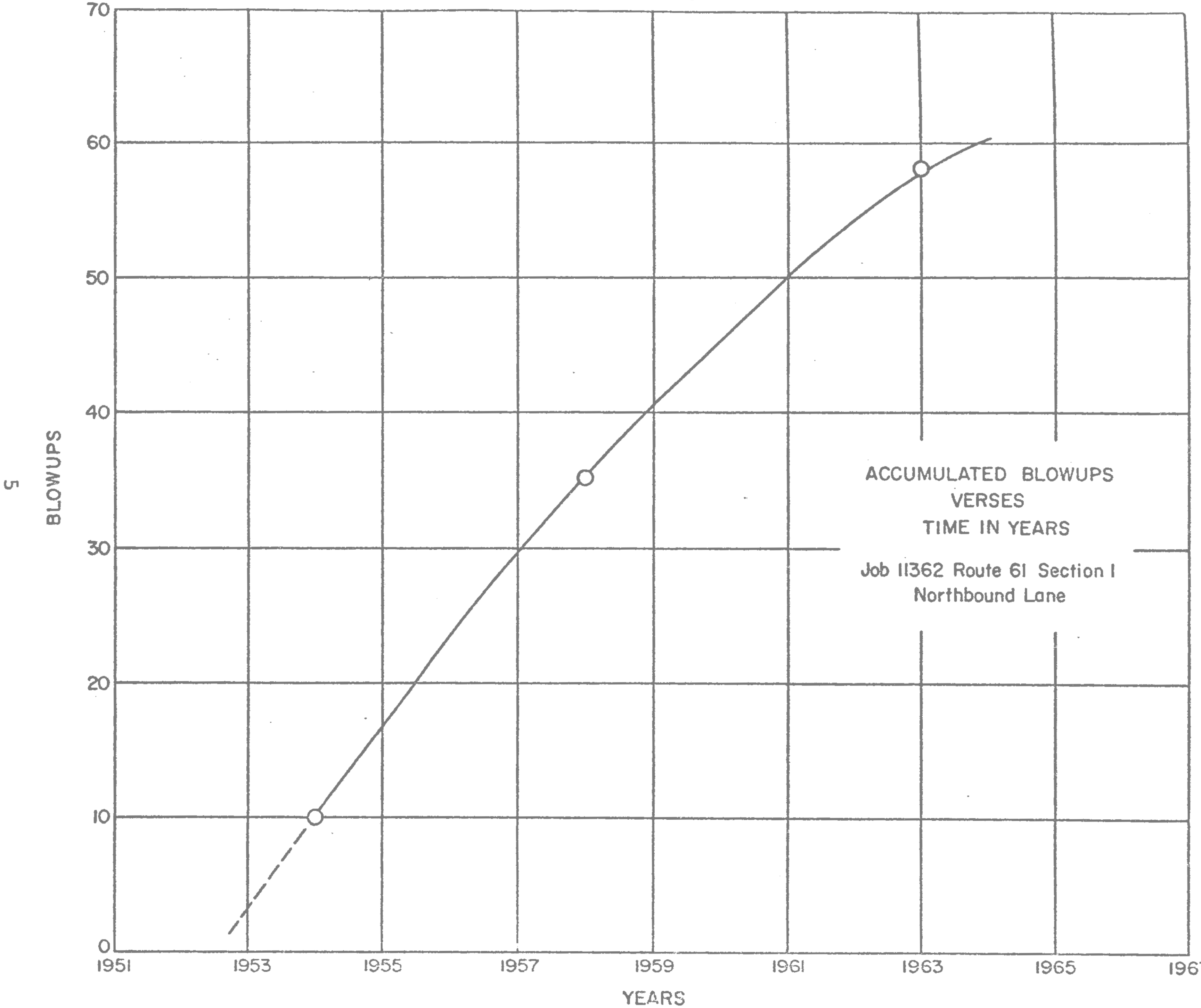
Prior to classifying the pavements into the different aggregate categories, an attempt was made to set up mathematical models involving time of construction -- with age of pavement -- in relation to blowups. No pertinent information was gained from this study which revealed the cause of blowups. Some useful information was gained in general from the study as to the rate of blowups vs. time, but for each different job and each coarse aggregate, the relationship would vary.

The two sections of pavement in Arkansas where blowups are most common are Route 61, Section 1, in the northbound lane, and Route 70, Sections 11 and 12, which is now I-30 Sec. 23. The curve in Figure 1, page 5, shows the history of these blowups on Route 61. The construction date was 1951, and Figure 1 shows that the first blowup period was in 1954 with a total of 10 occurring that year. The number of blowups is then accumulated to the years 1958 and 1963. Much maintenance was performed in the years from 1958 to 1963 to prevent these blowups. Extra expansion joints were sawed to a four-inch width to absorb expansion and relieve compression stresses. The effect of this type maintenance is reflected by the curve in Figure 1, which shows the continuation of the blowups at approximately the same rate as prior to 1958.

Several of the District Engineers have installed "bleeder" ditches or French drains along the edge of the pavements, with "bleeders" through the shoulders where excess moisture is under the slab. These District Engineers have realized they have a problem, and they know that unless the free water is drained from underneath the slab, the problem will be greatly increased. When the French drain or "bleeder" ditch is installed and the shoulder is resealed, no more excess water appears at the edge of the pavement. The District Engineer knows that this "bleeder" ditch will carry the excess water; what he can not determine is how much of the subbase will be lost through this ditch causing substantial damage to the base course or subbase. Also, the impact of traffic on the slab could cause pumping in and out of the ditch, and this would be unnoticed at the surface until the damage accumulated to the extent that it caused a slab failure. Some of these "bleeder" ditches and the pumping are shown in the Figures on page 6. Abstracts 1, 2, and 3 of the bibliography discuss and point out some of the problems involved by the presence of water underneath the slab.

The other route under discussion, Route 70, Sections 11 and 12, was constructed in 1954. Blowups started occurring in 1957 and 1958. A similar curve could be plotted for this route, but it would not be expected to "peak out" until 1965 or 1966. Route 70, Sections 11 and 12, had more blowups in 1964 than in 1963; it is anticipated that an even higher number will occur in 1965.

. IGURE I


70
60
50
40
30
20
10
0
BLOWUPS
1951
1953
1955
1957
1959
1961
1963
1965
1967
YEARS
ACCUMULATED BLOWUPS
VERSES
TIME IN YEARS
Job 11362 Route 61 Section I
Northbound Lane

ROUTE 70 - SECTION 11 - SBL

FIGURE 1

FIGURE 2

FIGURE 3

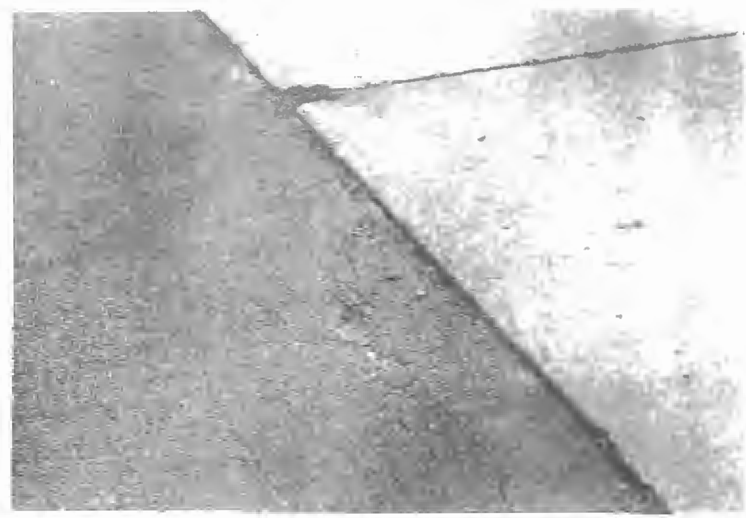


FIGURE 4

SLAB PUMPING

After the jobs were picked by the Project Director and broken down into the two groups of study, a method was derived by which a pavement of approximately five years old could be compared to a pavement approximately fifteen years old. This method was: divide the number of blowups by the length of the pavement in miles times the number of years since the date of construction. Since the blowups were inventoried in 1963, that date will be used as the base of all pavements' life in the study. As an example, a 10 mile stretch of pavement that was constructed in 1953 had 10 blowups as inventoried in 1963; then

$$\frac{10 \text{ blowups}}{10 \text{ miles} \times 10 \text{ years}} = 0.10 \text{ blowup per yr.-mi.}$$

These units will be used throughout the report to compare jobs in joint design, aggregate, and subgrade.

Tables I and II, page 8-9, show the jobs included in the study by construction job number and route and section. All information pertaining to the jobs is shown under the column headings, and the last column shows the blowups per year-mile as an indication of performance. In this study any job containing more than 0.01 blowup per yr.-mi. will be considered as giving service less than satisfactory. The source of coarse aggregate is listed with a numerical code, and the characteristics of this aggregate will be shown later in the study.

TABLE I

PROJECT STUDY DATA

Job No.	Dist.	Route & Section	Length	Yr. Const.	Aggr.	Source of Material	Joint Design	Blow-Ups	Blow-Ups Per Mi.	Yr. Miles	Blow-Ups Per Yr. Mile
6478	6	67-10 N.B.L.	4.20	1957	Cr.Sto.	1	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	25.20	0.00
6517	6	70-12 N.B.L.	9.05	1958	Cr.Sto.	1	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	45.25	0.00
6516	6	70-11 N.B.L.	8.31	1959	Cr.Sto.	1	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	33.24	0.00
6507	6	67-10 S.B.L.	5.84	1959	Cr.Sto.	1	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	23.36	0.00
6615	6	67-10 S.B.L.	7.34	1960	Cr.Sto.	1	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	22.02	0.00
6745	6	67-10 S.B.L.	4.20	1961	Cr.Sto.	2	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	8.40	0.00
6695	6	67-10 N.B.L.	7.34	1961	Cr.Sto.	1	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	14.68	0.00
6744	6	67-10 N.B.L. S.B.L.	1.68	1962	Cr.Sto.	3	No Exp., 45'-D. Cont. Jt.&15'-S. Warp, Mesh	0	0.00	1.68	0.00
TOTALS			47.96 Miles					0	0.00	173.79	0.00

Calculations:

Avg. Blow-up/mi. = 0.00

Avg. Pav. Age/mi. = $\frac{173.79}{47.96}$ = 3.62 yrs.

Avg. Blow-up/yr.mi. = 0.00

Note:
1. These calculations are based on a 1963 Blow-up Count and Age.
2. All the above jobs were designed on 4% Air-Entraining Content.

ABBREVIATIONS

Cr. Sto. ---- Crushed stone
Gr. ------ Washed gravel
Corr. ----- Corrugated
N.B.L. ---- Northbound lane
S.B.L. ---- Southbound lane
Exp. ----- Expansion
D. ------- Doweled
Cont. ----- Contraction
Jt. ------ Joint
S. ------- Sawed
Mesh. ----- Welded Wire Reinforcement

TABLE II

PROJECT STUDY DATA

Job No.	Dist.	Route & Section	Length	Year Const.	Aggr.	Source of Material	Joint Design	Blow-Ups	Blow-Ups Per Mile	Year Miles	Blow-Ups Per Year Mile
2331	2	65-20	14.83	1943	Gr.	4	20' Joints No Mesh	24	1.62	296.60	0.08
3351	3	67-1	1.36	1950	Gr.	5	200'-D. Exp. Jt. 20' Cont. Jt. Mesh	4	2.94	17.68	0.23
8257	8	7-14	3.07	1950	Gr.	6	200'-D. Exp. Jt. 20' Cont. Jt. Mesh	15	4.89	39.91	0.38
10406	10	63-9	5.67	1951	Gr.	7	400'-D. Exp. Jt. 25'-D. Cont. Jt. Mesh	26	3.90	68.04	0.38
11362	1	61-1 N.B.L.	7.19	1951	Gr.	8	400'-D. Exp. Jt. 40'-D. Cont. Jt. Mesh	58	8.07	86.28	0.67
8301	8	64-7	3.28	1951	Gr.	6	200'-D. Exp. Jt. 25' Cont. Jt. Mesh	2	0.61	39.36	0.05
10442	10	67-18	12.36	1952	Gr.	9	400'-D. Exp. Jt. 25' Cont. Jt. Mesh	37	3.00	135.96	0.27
11440	1	61-1 N.B.L.	13.06	1953	Gr.	8	400'-D. Exp. Jt. 40'-D. Cont. Jt. Mesh	38	2.91	130.60	0.29
3459	3	67-1	8.06	1953	Gr.	5	400'-D. Exp. Jt. 40'-D. Cont. Jt. Mesh	7	0.87	80.60	0.09
1276	1	1-6	4.31	1954	Gr.	10	No. Exp. Jt. 25'-Corr. Metal Cont. Jt. Mesh	0	0.00	38.79	0.00
3427	3	67-1	6.76	1954	Gr.	5	No. Exp. Jt. 40'-D. Cont. Jt. Mesh	38	5.61	60.84	0.62
6448	6	70-11 N.B.L. S.B.L.	11.01	1954	Gr.	11	No. Exp. Jt. 25'-Corr. Metal Cont. Jt. Mesh	7	0.64	99.09	0.07
6449	6	70-12 S.B.L.	9.05	1954	Gr.	11	No. Exp. Jt. 25'-Corr. Metal Cont. Jt. Mesh	9	1.00	81.45	0.11
6627	6	70-11 N.B.L. S.B.L.	6.08	1959	Gr.	11	No. Exp. 45'-D. Cont. 15'-S. Warp, Mesh	2	0.33	24.32	0.08
11540	1	61-1 S.B.L.	13.06	1960	Gr.	12	No. Exp. 45'-D. Cont. 15'-S. Warp, Mesh	16	1.23	39.18	0.11
11445	1	61-1 S.B.L.	7.19	1961	Gr.	13	No. Exp. 45'-D Cont. 15'-S. Warp, Mesh	5	0.70	14.38	0.35
11610	1	40-52 E.B.L. W.B.L.	16.26	1962	Gr. Sto.	14	No. Exp. 45'-D. Cont. 15'-S. Warp, Mesh	0	0.00	16.26	0.00
10623	10	67,21,22	11.51	1960	Gr.	15	No. Exp. 45'-D. Cont. 15'-S. Warp, Mesh	0	0.00	34.53	0.00
TOTALS			154.11					288		1,303.87	

Calculations:

$$\text{Avg. Blow-ups/Mi.} = \frac{288}{154.11} = 1.87$$

$$\text{Avg. Pav. Age/Mi.} = \frac{1,303.87}{154.11} = 8.46 \text{ yrs.}$$

$$\text{Avg. Blow-up/yr. mi.} = \frac{288}{1,303.87} = 0.221$$

Note:
1. These calculations are based on a 1963 Blow-up Count and Age.

2. All the above jobs were designed on 4% Air-Entraining, except Job 2331.

CHAPTER II
FIELD INVESTIGATION OF BLOWUPS

The blowup season started with the first blowup on May 20, 1964, on Route 70, Section 12. Many routine trips were made, driving over the pavements and investigating spalled areas and map-cracking. Reports of blowups from District Engineers all over the State began coming in by the first of June.

A study of each report was made, and by the 1st of July it was evident that all the blowups occurred in the middle of the afternoon, usually about 3:00 p.m., when the temperature was 90° F or more. These findings concur with those reported in this field by other states.

It was also determined from inspection that all blowups occurred at contraction joints or construction joints. Not one single blowup has occurred at a warping joint or expansion joint.

The two most troublesome routes, Route 61, Sec. 1, in eastern Arkansas, and Route 70, Secs. 11 and 12, in Central Arkansas, had completely different joint designs. Route 61 had 400-foot doweled expansion joints with 40-foot doweled contraction joints. Route 70 had no expansion joints and had a corrugated metal contraction joint every 25 feet. Both the pavements were reinforced with welded wire mesh. This was not continuous through either the 40-foot doweled contraction or the 25-foot corrugated metal joint.

In several cases, it was possible to arrive at the site of the blowup shortly after it had occurred. In these cases, the previously determined time of around 3:00 p.m. for blowups was further confirmed, among other data that was gathered. Upon arriving at a blowup site, a thermometer was laid down on the slab, and this reading was used as the slab temperature. The air temperature was obtained by holding the thermometer in the air at the blowup site.

While the temperatures were being taken, the maintenance forces usually removed some of the broken pavement, and the interior of the blowup could be observed. The interior was consistently the same except for one case. To begin with, the bottom portion of slab had always deteriorated to the extent that no mortar bond was apparent between the coarse aggregate, and the slab would be broken into pieces of varying sizes. (For examples, see figures 1, 2, & 3, page 11.) The bottom portion of the slab was saturated with water except in one case. The presence of water can be seen in figure 1, page 11, by the damping appearance of the concrete around the dowel bar. This bar was placed in the middle of the 10-inch slab, and it can be seen that this pavement section has about four inches of effective depth against any flexure stress. It was also apparent that infiltration from the joint and from the base or subbase was mixed with this deteriorated concrete.

When enough of the slab debris had been cleared, a moisture test was run on the base by use of the speedy moisture tester. A hand auger

ROUTE 61 - SECTION 1 - SBL

FIGURE 1

FIGURE 2

FIGURE 3

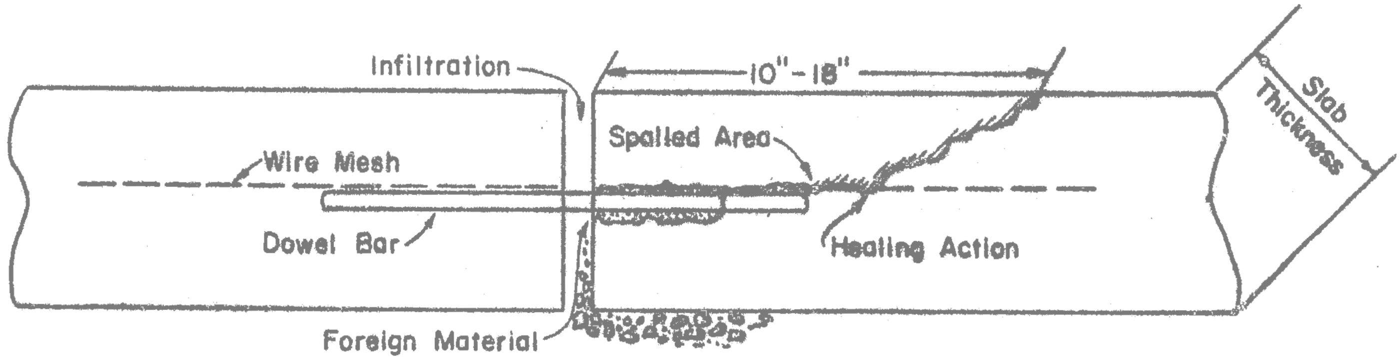
CASE NO. 2

was used to drill into the subbase and subgrade. A high water content was apparent in the subbase and subgrade as well as in the base course. When a base course was placed on a granular material, the water content was seldom high, as compared to the water content of the base materials placed over a non-granular subbase or subgrade.

In a blowup-prone pavement, the subgrade material could be classified as moderately impervious, with a medium high PI. Some of the temperatures and water contents are shown in table III, page 13, as found in the field investigations.

In early 1963, spalling at the corrugated metal joints was studied. This type of spalling was not too different from the blowups. As the Maintenance forces were repairing these spalled areas, several large pieces were gathered and returned to the laboratory for study. On the bottom portion of the spalled pieces, there was always a healing action. The spalled pieces would break out at the same depth as would the blowups, approximately four inches from the top of the slab. The healing action is easily identified in that the bottom surface of the pieces that have spalled out will be coated with a dry layer of cement which in the presence of water has leached out of the slab. When the slab breaks in this condition, its initial phase may not be noted as spalling. Map-cracking will usually be the first sign of trouble. Spalling much like blowups usually occurs around the contraction and construction joints; however, a contraction or construction joint that has spalled areas has not been noted to blow up. If the Maintenance forces elected to remove some of the larger pieces of slab that have spalled, as in Figures 2 and 4 on page 14, some spalled areas would show identical deterioration as in the blowup areas, except for the absence of water. From observation of the material that has infiltrated deep into the spalled areas, there must have been surface water through the contraction joint during the early phase of spalling action. There has also been observed some foreign material that has been pumped into the separation plane, as shown in the figure below:

FIGURE II



CROSS SECTION OF SPALLED AREA

TABLE III

Findings at Blowups that were Investigated immediately after they occurred.

Time Blowup Occurred	Temperature (F)		Moisture Content Percent	
	Air.	Slab	Base	Subgrade
2:45 PM	92^o	$104^o\pm$	13.4	18.4
3:00 PM	96^o	$116^o\pm$	15.6	22.4
3:00 PM	98^o	$115^o\pm$	17.2	19.0
3:00 PM	108^o	$125^o\pm$	14.0	16.0
3:00 PM	108^o	$125^o\pm$	13.0	15.0
11:30 AM	94^o	$108^o\pm$	15.2	18.7

ROUTE 70 - SECTIONS 11 & 12 - SBL

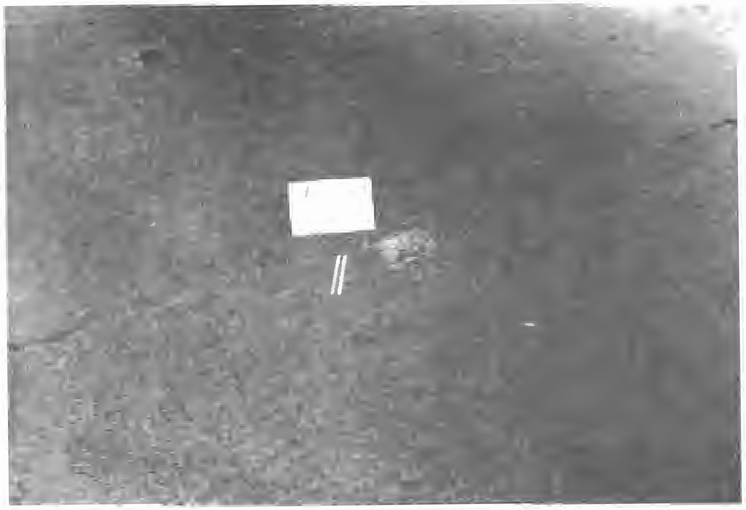


FIGURE 1

FIGURE 2

FIGURE 3

FIGURE 4

The type spalling discussed in the foregoing is relatively isolated, in that this could be attributed to joint placement. This type of deep spalling is associated with the corrugated metal joint, as it was vibrated into the pavement after it was poured. This vibration caused a highly differential stress in the continuity of the 25-foot slab. Trouble spots were located by the aid of a Swiss compression hammer; the pavement was profiled across the joints for soundness of the pavement. The results of such a profile are shown in Figure III, page 16. From observing the figure: The stress is about equal at each joint where the corrugated metal joint was vibrated into the pavement, whereas a decrease is shown at the center of the slab. The centerline in Figure 2, page 14, was a sawed joint; no spalling is evident along this centerline.

Another type of spalling that was observed is the pop-out type and is not associated with the joints. This type pop-out is generally associated with a cherty aggregate material, either in the fine aggregate or in the coarse aggregate. The field observations found no direct relationship between these pop-outs and blowups, except the fact that both will occur in the same section of pavement, which is to be expected, as will be pointed out later in the study under characteristics of materials.

Map-cracking as observed in this study ranged from hair-line surface cracking to much larger and deeper cracking. Map-cracking should not be confused with curing cracks. The map-cracking was observed to be characteristic of the pavements that contained a large number of blowups. It was observed that all pavements that had blow-ups contained map-cracking, but not all pavements that contained map-cracking had blowups.

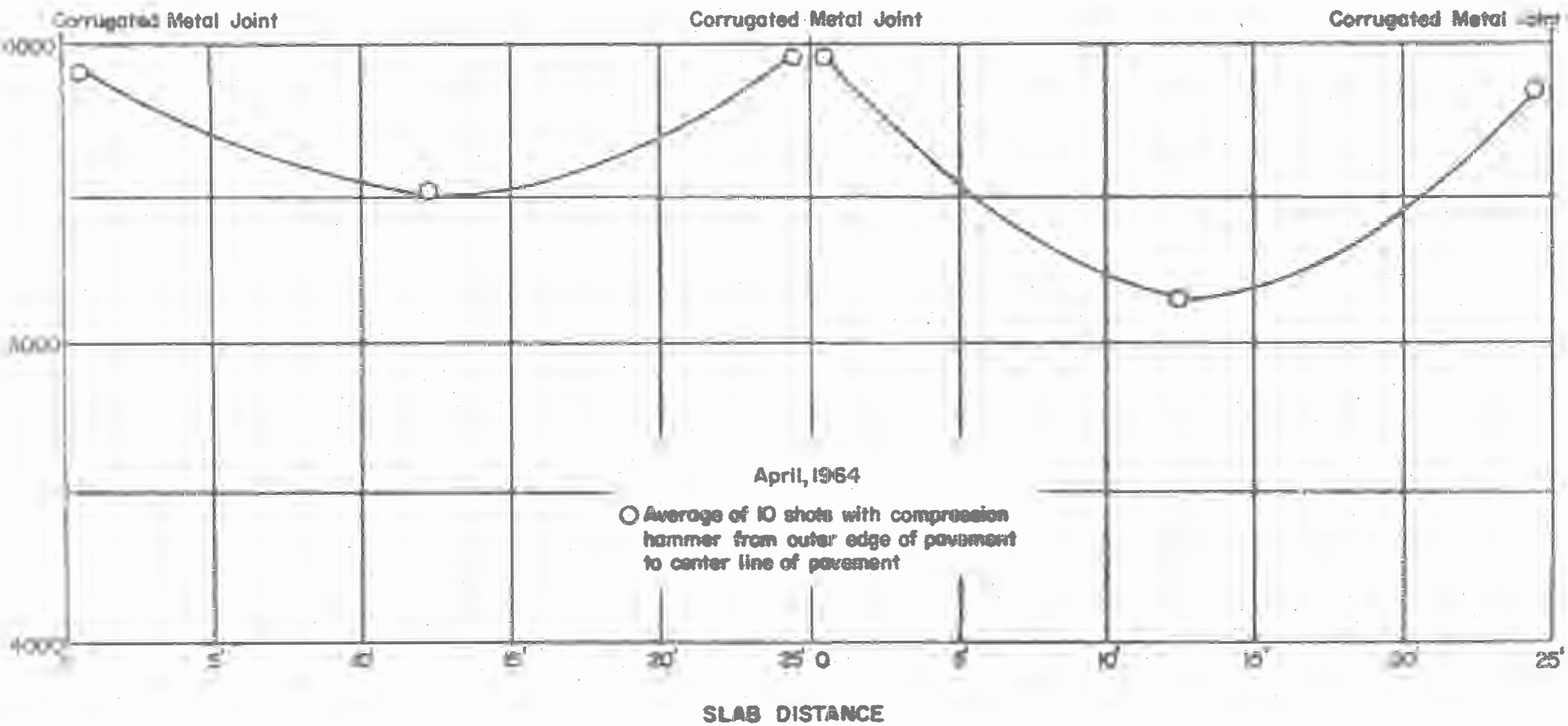
The structures that were built on these jobs using the same source of coarse aggregate were observed for spalling and map-cracking. Several of the bridge decks had signs of spalling along the gutter lines. The concrete in these spalled areas could be easily chipped out, and this concrete had the appearance of a dead mortar as in the other spalled areas. The occurrence of this type spalling in Arkansas is not too serious.

All cracks around the contraction and construction joints were studied, and an attempt was made to classify them as one of the following -- map, spalling, restraining, or curing cracks. All of these have been mentioned except restraint cracking. The restraint cracks occur when two adjoining slabs are undergoing additional compression from an excess of expansion in the slab. Any infiltration in this joint would cause an additional concentration of stress, and a crack on each side of the joint would result.

The pictures on pages 17 and 18 were made at a typical blowup site. On page 17, the typical four-inch portion can be seen in the raised position in figures 1 and 2. The blowup occurred on the entire width of the 24-foot pavement, and again the four inches of raised slab can be seen in figure 4 on page 17. The piece of concrete in figure 3, page 17, was hurled 95 feet from the joint at which the blowup occurred. This joint had no dowels; however, the wire reinforcement that was used

FIGURE III

COMPRESSION STRENGTH VARIATION



ROUTE 70 - SECTION 11 - SBL

FIGURE 1

FIGURE 2

FIGURE 3

FIGURE 4

CASE NO. 4

ROUTE 70 - SECTION 11 - SBL

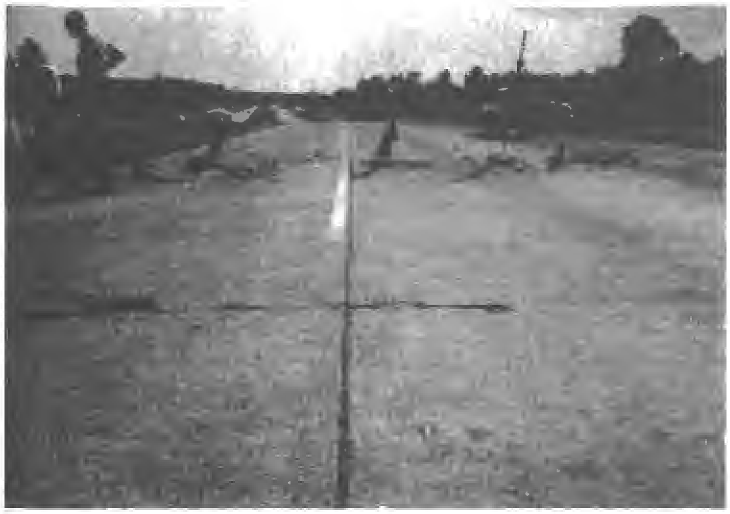


FIGURE 1

FIGURE 2

CASE NO. 4

failed in tension. In checking back with the laboratory, it was found that in testing this wire, it usually fails at about 90,000 p.s.i. Using this and the 95 feet that the piece of concrete was hurled indicates a tremendous force, very instantaneous. This joint was of the corrugated metal type.

Figures 1 and 2, page 20, illustrate poor drainage of the subgrade material in a blowup-prone pavement. These photos were made during a drought period. The water grass seen in the ditches illustrates a moderately impermeable subgrade material. A high water table is very characteristic of this area.

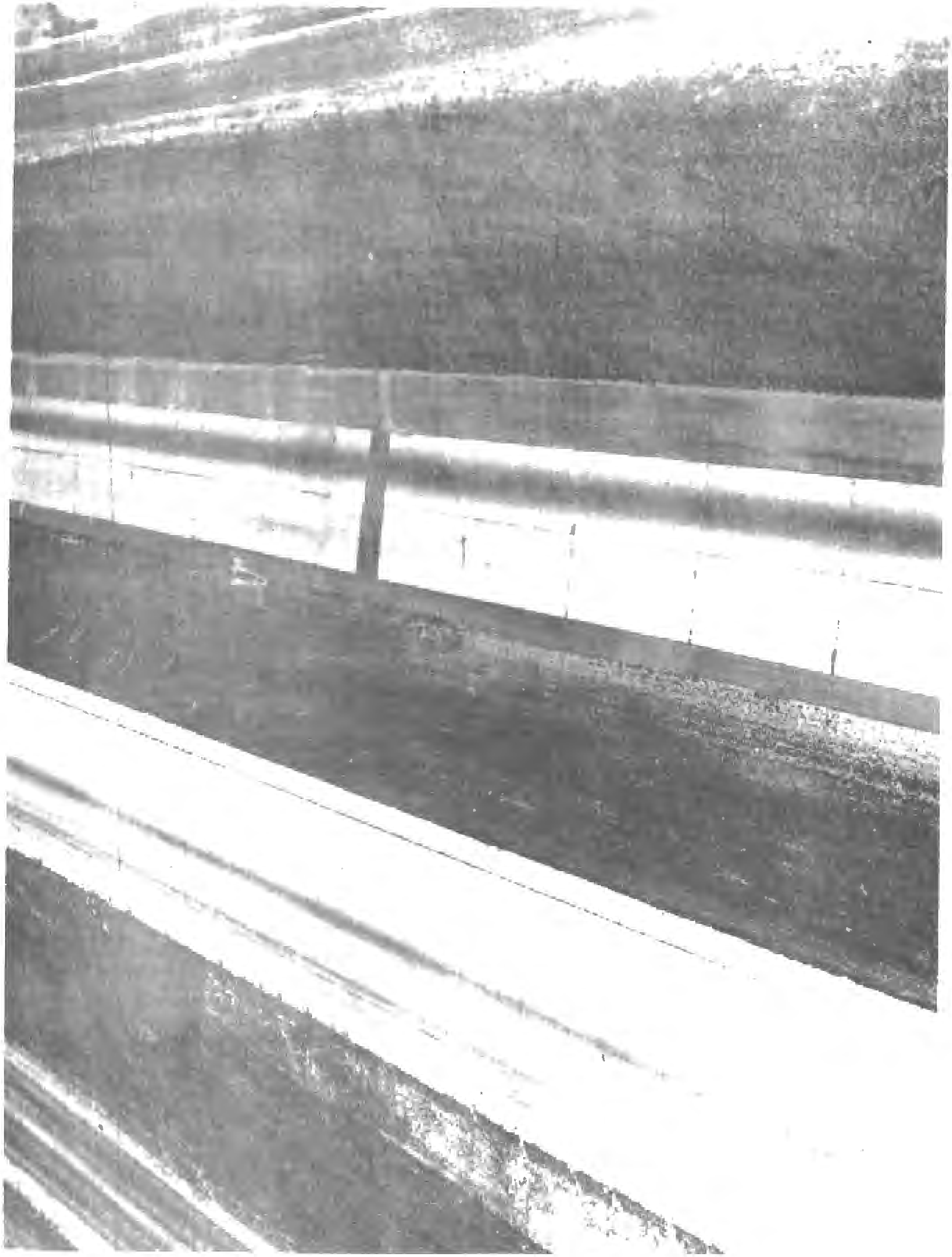
The photo on page 21 shows the comparison of two sections, with bleeding very evident in the right-hand section. The right-hand section has a granular base course, and the left-hand section has a crushed stone base course. The right-hand section has natural gravel as a coarse aggregate, and the left-hand section is constructed with crushed stone aggregate. The profile grade line is carried along the inside edge of both sections. This accounts for the bleeding being on the outside of the pavement edge. A blowup has occurred in the right-hand section.

ROUTE 61 - SECTION 1 - SBL

FIGURE 1

FIGURE 2

POOR DRAINAGE EXISTENCE



COMPARISON OF PAVEMENT CONDITIONS

CHAPTER III
DETAILED FIELD AND LABORATORY STUDIES

The initial work outline for this study was mainly concerned with study of records from the job files and observations to be made in the field. Some cores were to be drilled and the characteristics of each core compared and correlated to the performance of the pavement. When the records study was completed, a revised work outline was made, and more laboratory and field work were included.

The cores were drilled in accordance with the work outline, and the determination of the drilling location was aided by the use of the Swiss hammer. When the cores were drilled, some of the considerations were: the compression strength, as determined by the hammer; the general appearance of the pavement; and location on vertical alignment, such as a cut or fill and the sag or crest of a vertical curve. In the laboratory, the cores were allowed to air-dry for 28 days at 76° F. They were then weighed and measured precisely for the beginning of the laboratory work. Since the areas of study had previously been grouped into the two classes of aggregate, (1) washed gravels, and (2) crushed stone, the cores were grouped accordingly, and placed in a water bath with the water level one-half inch deep (final depth). Absorption measurements were made at 4, 8, 24, and 48 hours, respectively. The study was conducted in this manner, based on the assumption that the rate of absorption in a rigid pavement was more harmful than the amount over a given period of time. The previous field investigations had revealed much evidence of pavement damage due to differentials in stress between the top and bottom of the slab.

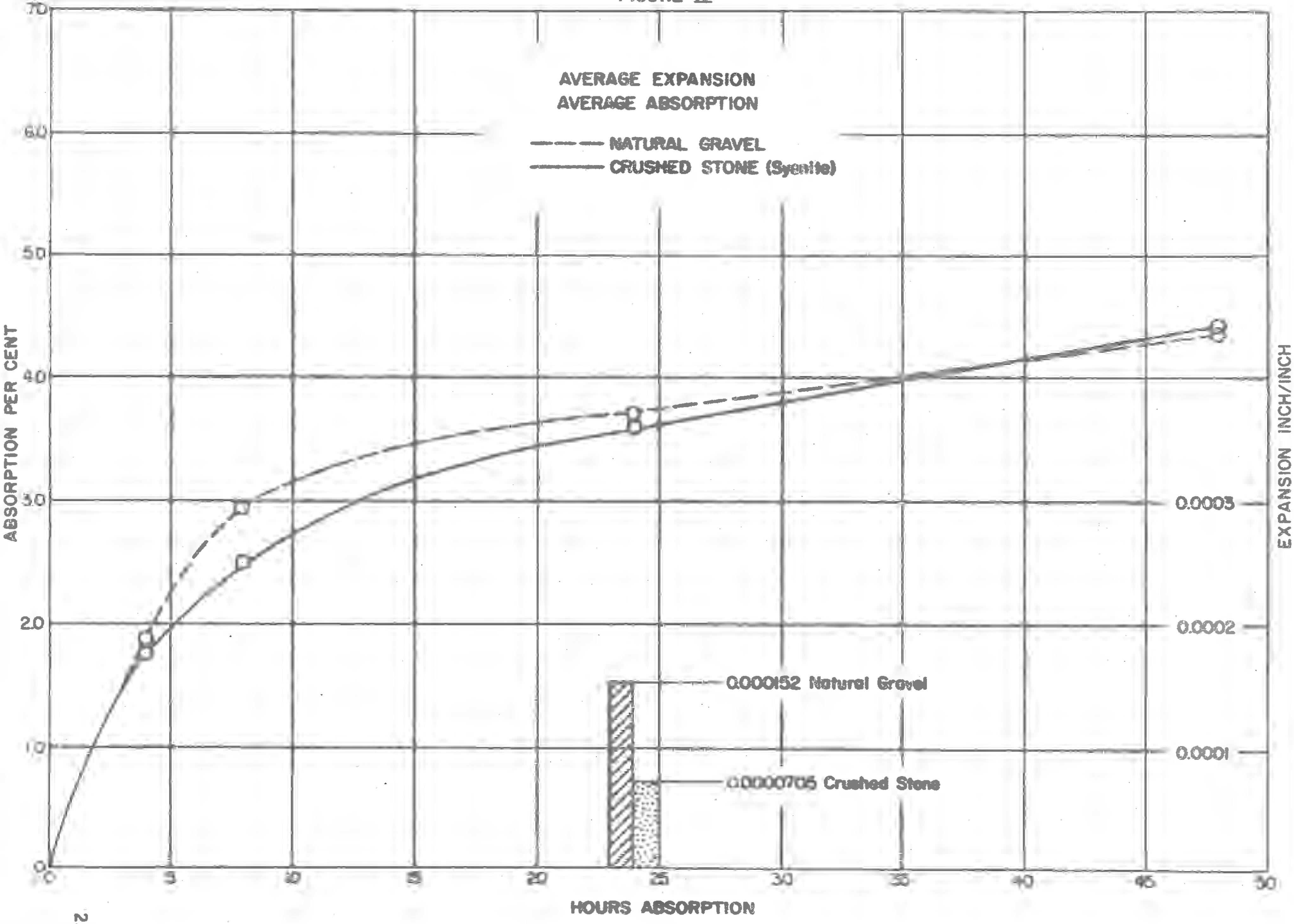
The expansion due to moisture was computed at the 24-hour absorption. No additional expansion tests were made beyond the 24-hour absorption period. The expansion due to moisture for the washed gravel aggregate concrete was more than double that of the crushed stone concrete. The absorption rate was slightly greater for the washed gravel, but after about 40 hours' absorption, the two curves are almost equal. The results of this testing are shown graphically in Figure IV. Not all the jobs included in the study have been core-drilled; for the results of research to date, further core-drilling would give repetitious results.

The remaining jobs may be drilled at a future date for additional information pertaining to soil subgrade and base course material with respect to subgrade drainage. The expansion due to temperature change on the cores was negligible. The cores were placed in large, metal gas-fired ovens and heated to 150° F for a period of eight hours, which was a 74° F increase in temperature over room temperature.

Later in the stage of research, some beams were made for the purpose of studying thermal fatigue and expansion. The average expansion for concrete made with crushed stone was 0.000004*, while the average coefficient for concrete made with washed gravels was 0.000005*. The procedure used on the expansion test for the beams was as follows: The beams were cast in the field on various paving jobs, of which all had similar mix designs, using a 5.5 bags per yd.^{3} and 4% air-entrainment. The beams

*Unit expansion/of

FIGURE IV



AVERAGE EXPANSION
AVERAGE ABSORPTION
NATURAL GRAVEL
CRUSHED STONE (Syenite)
0.000152 Natural Gravel
0.0000705 Crushed Stone
ABSORPTION PER CENT
EXPANSION INCH/INCH
HOURS ABSORPTION
0.0003
0.0002
0.0001
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
5
10
15
20
25
30
35
40
45
50

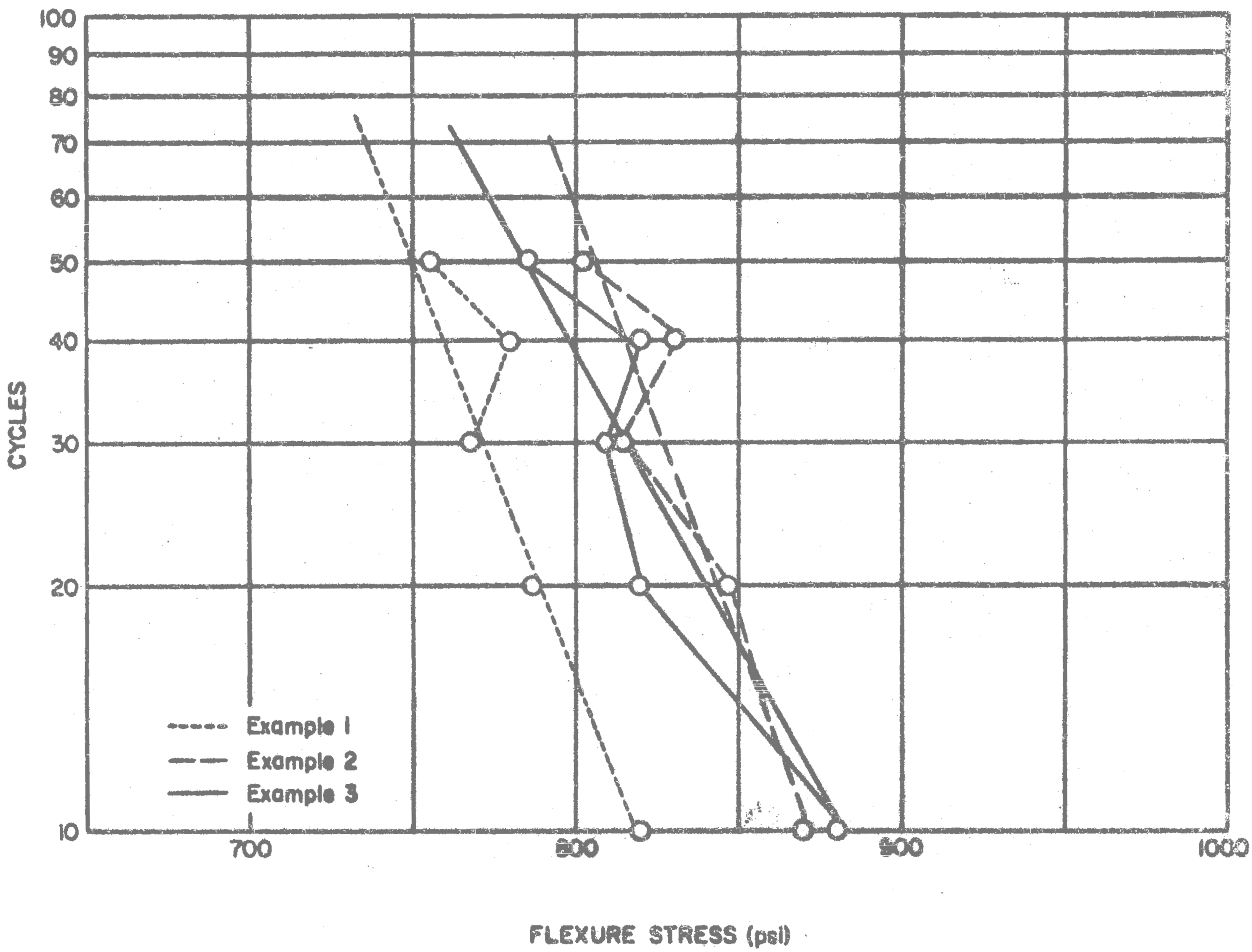
were cured in the field for seven days in the same manner as the construction test control beams, which consists of covering the beams with sand. The beams were then brought to the laboratory and moist-cured for 28 days. The beams were then air-dried to room temperature (76° F) for 10 days, and then calibrated by use of brass tacks that were set at the time the beams were made. Precision calipers were used for the calibration of the brass reference tacks. The beams were then heated in electric-fired ovens to 150° F, which again was a 74° rise in temperature. The result of this study is in line with other reported coefficients (11).

The thermal fatigue studies were very uniform and were conducted on the same beams as were the expansion tests. The beams were placed in the electric-fired ovens and heated for eight hours and cooled for twelve hours. After each ten cycles of such heating and cooling, the beams were placed in hydraulic testing machine, and a 300 psi pressure was applied while using the compression hammer to determine the flexure stress. This was repeated until a stable flexure strength was gained. The stabilized flexure stress on all beams after being subjected to this testing was still safe for a rigid pavement design, the lowest point being approximately 720 psi.

Figure V, page 25, shows the results of a thermal fatigue test. Curves 1 and 2 are carried out to 50 cycles of heating and cooling, and a definite trend of strength loss is shown. The points that are shown at 40 cycles were taken when the beams were removed from the oven and allowed to cool. The remaining points are test points made on the beams immediately after removal from the oven. The beams represented by curve 3 were subjected to wetting and drying. This test was conducted by submerging the beam in a water bath for eight hours and then drying for 16 hours in a gas-fired oven at 100° F. The 40 cycles point on curve 3 was made when the beam was in a heat-dry condition. All other points were made when the beam was wet.

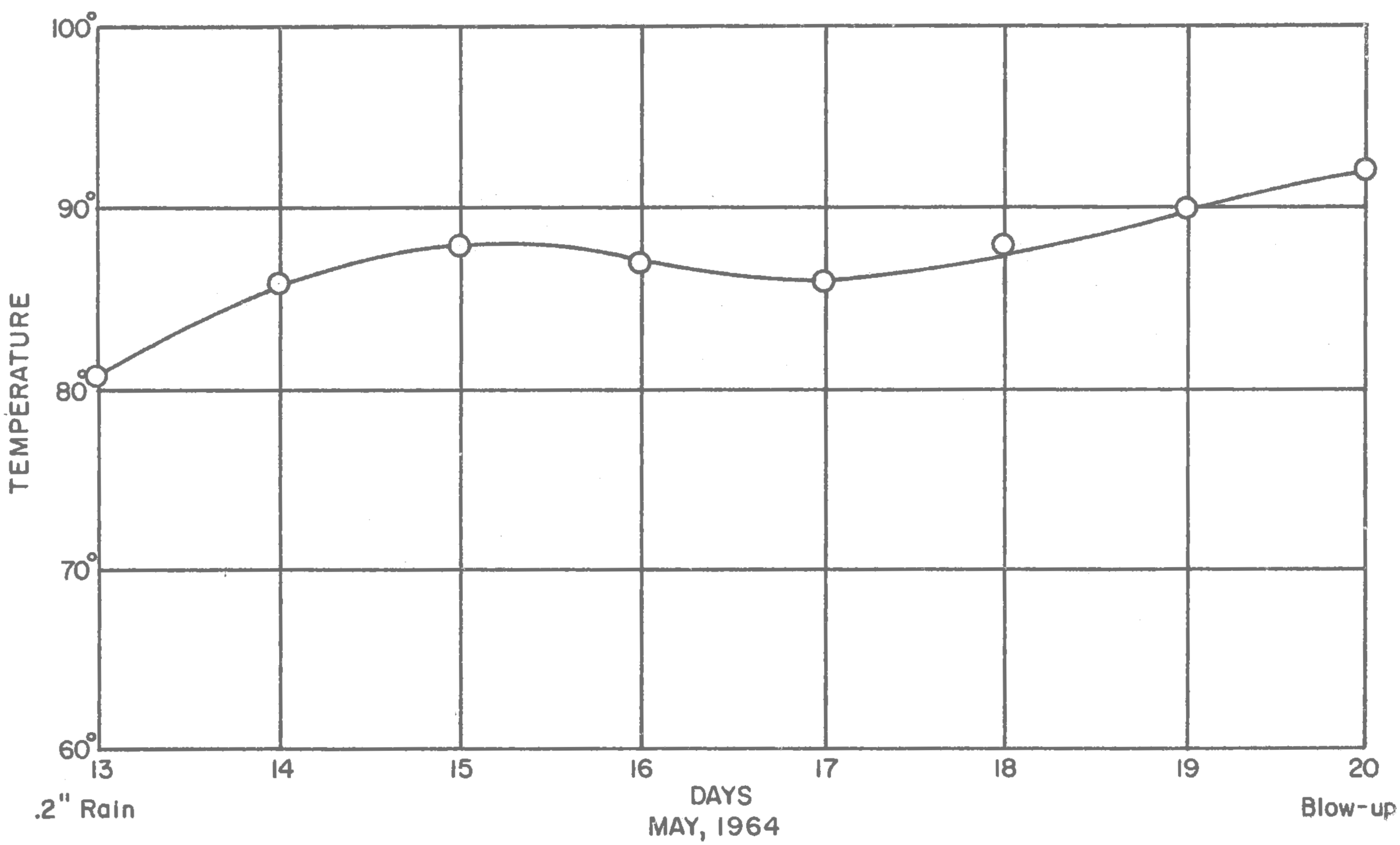
When the investigation of a blowup was made, all available data was collected in an attempt to establish any trend in air temperature or rainfall pattern as the cause of the blowups. Figure VI shows the temperature trend leading into a blowup. The temperature continued to rise after the blowup occurred, but no more blowups were reported in this area. All other blowups would have a curve similar to this; however, temperature trends of this nature did occur very frequently and no blowups were reported. Therefore, it can be said that blowups could not be predicted from a steadily upward trend in temperature.

FIGURE V



100
90
80
70
60
50
40
30
20
10
CYCLES
700
800
900
1000
FLEXURE STRESS (psi)
Example 1
Example 2
Example 3

FIGURE VI


100°
90°
80°
70°
60°
TEMPERATURE
13
14
15
16
17
18
19
20
.2" Rain
DAYS
MAY, 1964
Blow-up

The moisture contents of two different pavements, along with the densities, were studied by use of the Lane-Wells Road Logger, courtesy of the Lane-Wells Company, Houston, Texas. The two studies are shown graphically on pages 28 and 29. The moisture curves in both figures are of the utmost importance in the study.

A third road was logged which is not shown in this study, and the results were the same as those on page 28, with the moisture content being about 10 lbs./ft.^{3}. The two pavements with the lower moisture contents are relatively maintenance-free, while the pavement that contained the higher moisture content, about 13 lbs./ft.^{3}, has given considerable trouble. The writer does not attribute the entire maintenance cost, however, to the high moisture content; but it has caused the larger percentage of the cost in pavement blowups and pavement deteriorations. By observing the moisture curve on page 29, it can be noted that the moisture content is also very inconsistent, while that on page 28 is fairly constant.

Reference is made to the expansion bar on page 23 to estimate the damage that could be expected with a high moisture content in the pavement. The two curves under discussion show 7.12 and 9.76, percentage-wise, while the absorption shown on page 23 did not gain more than 4.4% at the end of the test. Fresh concrete, assuming 5.5 gallons of water per sack with a 5.5 bag mix, would contain about 9.3 lbs./ft.^{3} moisture, or about 6.6%, which means that the pavement has more moisture now than it did when it was poured. The moisture would be considerably greater on the bottom than on the top; therefore the greater influence due to this excess moisture would be in the bottom of the slab.

The low points in the moisture curve on page 29 represent the joints. The equipment was operating on a three-second time lag; therefore, the locations of the joints would be 7.5 feet behind the tick marks at the bottom of the page which correspond to the density curve.

The aggregate characteristics are shown in figures VII and VIII for comparison.

FIGURE VII
ROUTE 70 SECTION 12
N.B.L.
CRUSHED LIMESTONE PAVEMENT
AGGREGATE

Absorption 0.53% Sp. Gr. 2.61, Porosity 1.38%

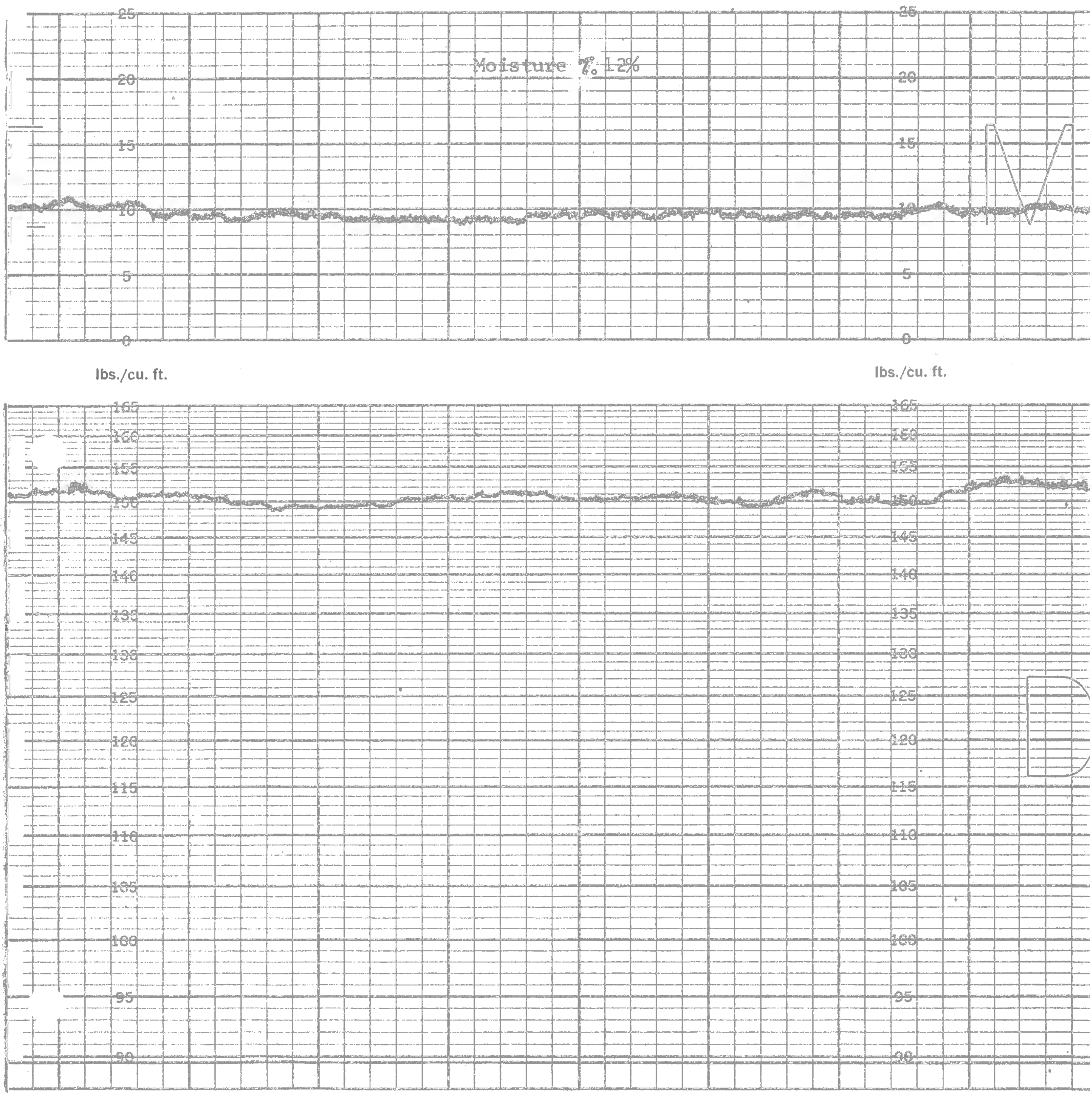
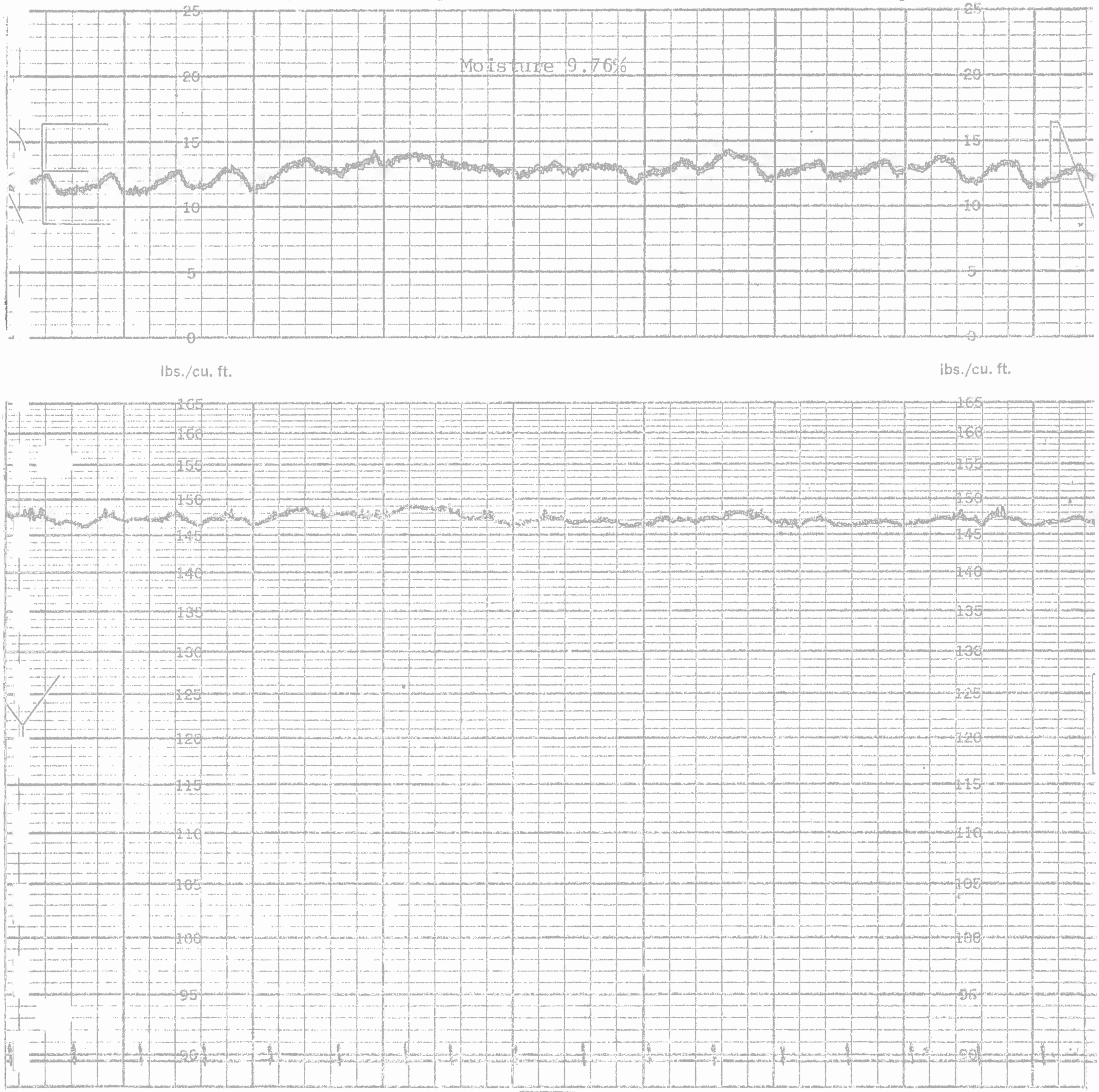


FIGURE VIII
ROUTE 70 SECTION 12
S.B.L.
NATURAL GRAVEL PAVEMENT
AGGREGRATE

Absorption 1.35%, Sp. Gr. 2.53 Porosity 3.4%



CHAPTER IV
ANALYSIS OF COARSE AGGREGATE

The general inventory and detailed studies, along with the field observation, indicated that the primary source of a blowup-prone pavement lies within the type of coarse aggregate. The characteristics of each aggregate were studied under this phase for any possible correlation between the blowups and the coarse aggregate. Earlier in the study, the aggregates were separated into the two groups, crushed stone or washed gravel. (It was observed that all pavements in which crushed stone was used as the coarse aggregate no blowups had occurred.)

The only valid correlation that could be developed when both types of aggregate were combined for the analysis was the porosity relationship with the blowups per year-mile. This relationship is shown in figure IX, page 31. The writer is of the opinion that no one characteristic can be said to cause a blowup, but several characteristics, including design and subgrade characteristics, acting together cause such a pavement failure. The correlation established between coarse aggregate porosity and blowups per year-mile is considered the major cause. From the maintenance view, if a pavement were constructed from the lowest possible porosity aggregate, and the warping joints or contraction joints were not properly maintained, pavement failures would be expected to occur through infiltration. At the same time, if the joints were spaced at excessively long intervals, pavement failures would be expected to occur. An endless list could be compiled of factors that would influence a pavement to the stage of a blowup.

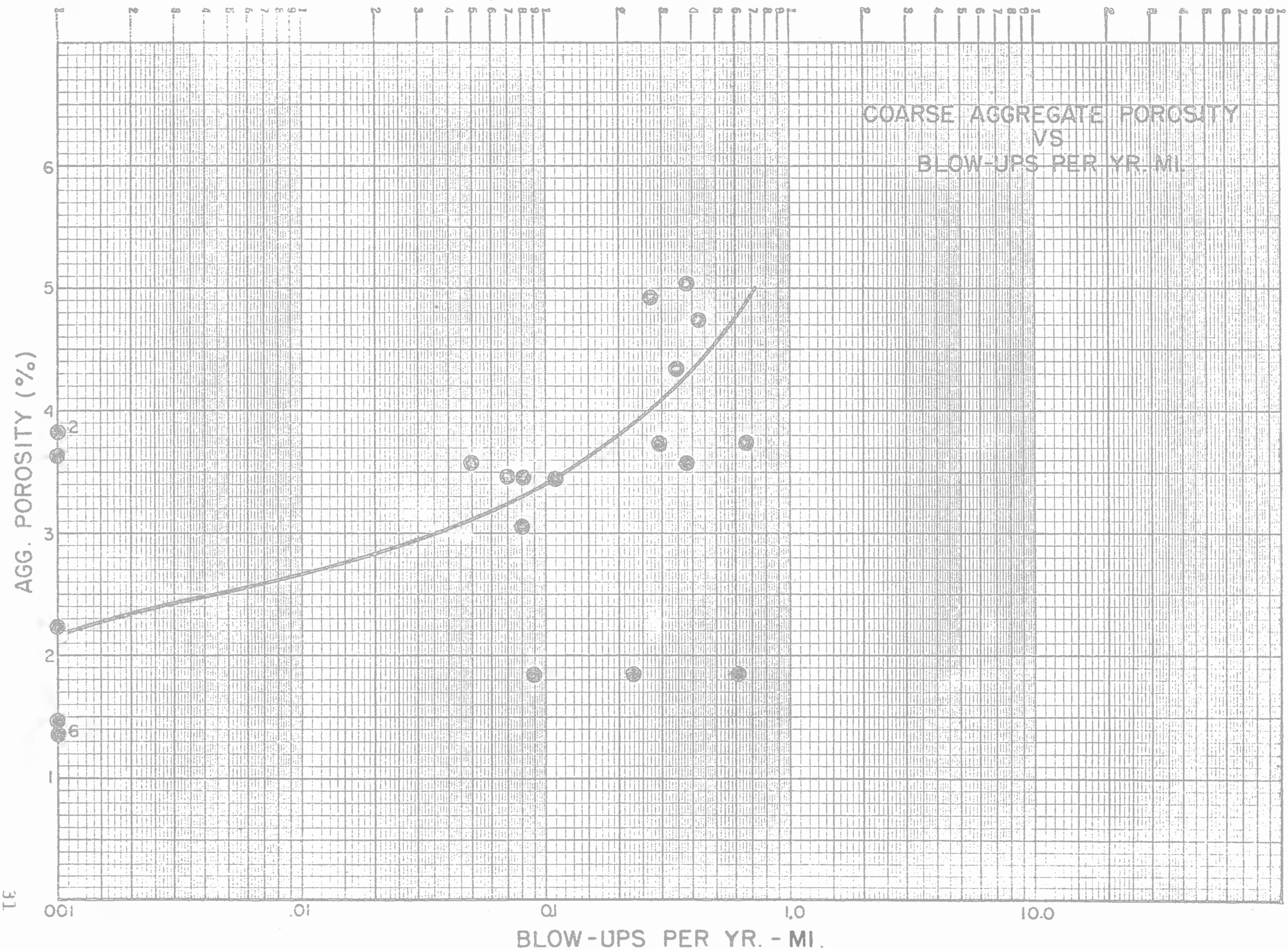
Some states have gone into much detail studying aggregate characteristics with respect to durability. The Joint Highway Research Program at Purdue University has made many studies of this nature. The report, "Deleterious Constituents of Indiana Gravels," (6), presents studies of durability, using heavy media separation for aggregate used in experimental design. It was concluded that greater durability is obtained as the specific gravity at which the separation occurs is increased.

The study of "The Effects of Quick Freezing on Saturated Fragments of Rocks" by the Kentucky Department of Highways presents a very thorough study of aggregate characteristics. The authors conclude that porosity could be used as an index to durability of aggregate.

Durability is associated with blowups, as noted in the field investigation, by the deterioration of the bottom portion of the slab. This deterioration is due to aggregate and its relation to the slab function. It has already been stated that no blowups occurred other than at the joints, and no deterioration of the bottom portion of the slab was found except at the joints. The writer feels that durability of aggregate and pavement blowups are somewhat one and the same problem.

Referring to our standard specifications in the analysis of coarse aggregate, we have the abrasion test, the soundness test, deleterious material and decantation. The absorption test (24 hours) and the specific gravity are run on all samples; however, no requirements are

FIGURE I


COARSE AGGREGATE POROSITY
VS
BLOW-UPS PER YR. MI.
AGG. POROSITY (%)
6
5
4
3
2
1
2
6
.001
.01
0.1
1.0
10.0
BLOW-UPS PER YR. - MI.

specified. Some forms of deleterious materials meet the abrasion and soundness test requirements, and in this case the District and Construction Engineers, et al., have recourse to another section of the specifications, "that the aggregate will be hard and durable." To eliminate pavement blowups or to minimize them, the State must begin with a more rigid specification.

Once the non-durable aggregate is barred by the specifications, Design and Maintenance are in the responsible position for the prevention of pavement blowups. To eliminate maintenance, the field observations and studies indicate that contraction and warping joints must be sealed at all times. Failure to seal these joints results in infiltration of the joint and also when heavy rains occur the joint fills with water, and the high-porosity aggregate then becomes the "number one" factor. The Design Section has always been faced with the problems of a high water table and a lack of adequate materials. These problems will always be present, even in the generations to come. The design of a rigid pavement should be correlated with aggregate characteristics, to include type of subgrade, climate, length of joint spacing, and permeability of the base, subbase, and subgrade.

All the factors above are directly related to the coarse aggregate porosity and could be explained by the curve in figure IX, page 31. The porosity was computed by the equation:

$$P = \frac{A}{100 + A} \times \text{Sp. Gr.} \times 100$$

where, P = porosity in percent

A = 24-hour absorption

Sp. Gr. = bulk specific gravity of the aggregate

A summary of the characteristics of 18 sources of aggregate is presented on page 33. Four of these 18 sources are out-of state aggregates that were incorporated into pavement jobs in recent years. The performance of all aggregates is also shown. The curve on page 31 was plotted using this data. Similar curves were attempted using specific gravities and absorptions shown on the data sheet, and no direct correlation existed.

SOURCE OF MATERIAL	BLOWUPS PER YR. MI.	PERCENT 24-HOUR ABSORPTION	BULK SP. GR. HEAT DRY	POROSITY PERCENT
1	0.0	0.53	2.61	1.38
1	0.0	0.53	2.61	1.38
1	0.0	0.53	2.61	1.38
1	0.0	0.53	2.61	1.38
1	0.0	0.53	2.61	1.38
1	0.0	0.53	2.61	1.38
2	0.0	1.50	2.57	3.80
3	0.0	0.86	2.63	2.24
4	0.08	1.23	2.54	3.09
5	0.23	0.71	2.59	1.83
6	0.38	1.45	2.48	3.54
7	0.38	2.12	2.42	5.02
8	0.67	1.48	2.51	3.73
6	0.05	1.45	2.48	3.54
9	0.27	2 02	2.49	4.91
8	0.29	1.48	2.51	3.73
5	0.09	0.71	2.59	1.83
10	0.0	1.53	2.53	3.80
5	0.62	0.71	2.59	1.83
11	0.07	1.35	2.53	3.47
11	0.11	1.35	2.53	3.47
11	0.08	1.35	2.53	3.47
12	0.41	2.20	2.45	4.73
13	0.35	1.75	2.51	4.33
14	0.0	1.45	2.53	3.62*
15	0.0	0.57	2.58	1.46*
16	- **	0.46	2.69	1.23*
17	- **	0.96	2.65	2.52*
18	- **	1.74	2.50	4.06

* Sources of coarse aggregate that was obtained outside the State of Arkansas and used in rigid pavements.

** Blowups not inventoried on these jobs as they are more recently constructed ones.

SUMMATION OF STUDY

1. Blowups start occurring in a pavement about four years from the construction date.

2. Blowups occur most generally in the afternoon about 3:00 p.m., eight to fifteen days after a rain.

3. Porosity of the coarse aggregate could be used as an index to the expected performance of a rigid pavement with a consistent joint design.

4. No blowups occurred at the sawed warping joints.

5. Frequency of blowups was less in the shorter slabs than in the longer ones.

6. The flexure strength showed a decrease under the thermal fatigue test; however, the resultant strength was still acceptable.

7. Blowups occurred more frequently where the pavement was laid over a moderately permeable subgrade, which had medium-high plasticity index.

8. Blowups seem to be caused by a combination of temperature and moisture in the concrete slab.

RECOMMENDATIONS

1. It is recommended that consideration be given to establishing a durability index from porosity, with a maximum allowable of 3.0%, on coarse aggregate used in concrete pavement. The only exception to this is in recommending that the contractor furnish a record that the material was used elsewhere and no blowups or spalling have occurred in six (6) years from the date of construction. This would also be acceptable.

2. A trend toward the reduction of joints is recommended, and with the continued use of the joints, the design should be based on the aggregate used in the pavement and the type of reinforcing.

3. Although most of the projects in this study contained dowel bars, it was observed that those structures without dowel bars were not undergoing extensive failures due to the absence of these bars. It is recommended that additional study be conducted in this area, with emphasis on improved capabilities of load carrying bases and subbases.

4. There has been much emphasis on the relationship of subsoil moisture to problem concrete pavements; it is recommended that more research be conducted in this area.

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3. Kuhlman, R. H., "Evaluation of Joint Practices in Concrete Pavements in Arkansas," unpublished paper, 1958.

4. Woods, K. B., Sweet, H. S., and Shelburne, T. E., "Rigid Pavement Blowups Correlated with Source of Coarse Aggregates," Proceedings, Highway Research Board, Vol. 25, 1945.

5. McKesson, C. L., "Study of Asphalt Treated Bases and the Importance of Vertical Moisture Differentials," Proceedings, Highway Research Board, Vol. 24, 1944.

6. Lewis, D. W., "Deleterious Constituents of Indiana Gravels," Joint Highway Research, Purdue University, 1953.

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14. Highway Research Board Special Report 61A and 73.

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(1) R. J. Wig, Chairman, Committee on Expansion and Contraction of Concrete Roads, Proceedings, National Conference on Concrete Road Building, p. 53 (1914).

"All tests indicate that the effect of moisture content is very much greater than the effect of temperature change and may be sufficient to cause a stress in concrete opposite to that caused by normal temperature range," (p. 24).

"Also if the concrete road is subject to a heavy rain for a considerable length of time the bottom will be exposed to moisture in the ground for a longer time than the surface and the surface will be more or less dried due to exposure to the sun and consequently tend to contract while the bottom would still be expanding," (p. 24).

The data contained in the Wig report shows slabs laid in June, during warm weather decreased in length due to moisture loss until August, then elongated until October presumably due to temperature, then continued to increase in length during the winter months until April, with a mean temperature dropping from about 60° F. in October to about 25° F. in midwinter. As the effect of a lowered temperature should have resulted in a shortening of the slab, the expansion is obviously due to moisture increases. The next year the slabs shorten as a result of moisture loss.

(2) K. B. Woods, H. S. Sweet and T. E. Shelburne, "Rigid Pavement Blowups Correlated with Source of Coarse Aggregates," Proceedings, Highway Research Board, Vol. 25, 1945.

These gentlemen have shown that the type of subgrade has a marked effect on the durability of concrete made with certain aggregates. Based on the conclusions drawn from this paper, not enough is known as to how to obtain low moisture contents in granular bases over impervious soils. More data showing moisture contents in subbases of various designs, and if possible, moisture contents in the overlying concrete, is needed in order to intelligently design granular subbases that will keep the concrete pavement relatively dry.

(3) C. L. McKesson, Director, Engineering and Research, American Bitumuls Company, HRB Proceedings, Vol. 24 (p. 466-477)

In this paper, Mr. McKesson recalls to the reader a number of possible causes for distortion in concrete pavement slabs.

a. External forces resulting from:
   1. Non-uniform soil swell caused by entrance of water into the subgrade soil.

   2. Non-uniform soil shrinkage caused by loss of moisture from the subgrade soil.

3. Non-uniform soil swell caused by the action of frost.

4. Flow or creep of the subgrade soil.

b. Internal forces resulting from:

1. Vertical temperature differential in the slab.

2. Vertical moisture differential in the slab.

3. Unequal deposition of crystalline matter in the top or bottom of the slab.

4. Unequal hydration of the cement in top and bottom of slab.

This paper emphasizes the probable importance of vertical moisture differentials in the concrete slab and moisture differentials as a direct and indirect cause of warping and pavement failures.

Several rigid pavements were constructed in the City of Oakland, California, over swelling clayey soils; some of the pavements were constructed over asphalt-treated bases while others were placed over non-treated bases on identical subgrades. The laboratory tests and field performance show that serious permanent warping can occur as a result of a slab resting on a saturated subgrade or subbase while the top is exposed to evaporation from air and solar heat. On the other hand, pavements that were constructed over asphalt-treated bases showed no warping, and little or no maintenance was required for the 10-year observation period. The treated bases also supplied ample support for non-doweled slabs, which was a reduction in initial cost.

With the asphalt-treated bases the leakage from cracks and joints cannot pass through the base to the subgrade to produce swelling or softening under the joints or cracks. The field investigation of untreated bases showed that moisture content is much greater in the subsoils under the joints than at the center of the slab.

From this report, the following recommendations and conclusions were made:

Recommendations -

1. That the thickness of the treated base be a minimum of 4" for light traffic and 6" for heavy traffic. If the designing engineer believes more thickness is required than that of the slab plus the recommended thickness of the treated base, then additional selected material should be placed on the subgrade.

2. That the treated base be carried out under shoulders or well beyond the edge of the slab to prevent moisture loss from subsoil under the edges of pavement.

Conclusions -

An emulsified asphalt-treated base:

1. Insures uniformity of moisture content beneath the concrete slab.

2. Substantially eliminates warping and contraction of concrete pavement insofar as these are due to moisture.

3. Provides uniform and continuous support under slabs and at joints and cracks.

(4) Deleterious Constituents of Indiana Gravels

D. W. Lewis, Research Engineer, and Edwards Venters, Research Assistant, Joint Highway Research Project, Purdue University. 1953.

This paper reports results, ranging from good to poor, of tests on Indiana gravels in field performance in Portland cement concrete. The gravels were separated by liquid flotation into various specific-gravity ranges. The gravel fractions thus obtained were tested to determine absorption, specific gravity, degree of saturation, lithologic composition, and durability of air-entrained concrete subjected to freezing and thawing.

The results show that the principal deleterious constituents of Indiana gravels are sandstones and cherts. They are characterized by low specific gravity, high absorption and degree of saturation, and produce nondurable concrete when used as the coarse aggregate. The concrete durability can be improved by heavy media separation of the aggregate. Greater durability is obtained as the specific gravity at which the separation is made is increased.

Conclusions:

1. The deleterious constituents of the gravels consist principally of sandstones and cherts, with lesser amounts of badly weathered, calcareous, igneous, and metamorphic rocks.

2. These nondurable particles are characterized by low specific gravity, high absorption and degree of saturation, and poor durability in concrete subjected to freezing and thawing.

3. They can be separated from the durable particles by heavy-media separation techniques.

4. The durability, both in the field and in the laboratory, of concrete made with the gravels from various sources is dependent upon the quantities of low specific gravity material contained in the aggregate.

5. Heavy-media separation can be used to improve the durability of concrete made with these gravels.

6. The durability of concrete made with the separated gravel fractions increases as the specific gravity of the gravel is increased.

(5) John W. Scott and George R. Laughlin, Research Engineers, A Study of the Effects of Quick Freezing on Saturated Fragments of Rocks, Kentucky Department of Highways Research Report, 1964

In the letter of transmittal for this report, James H. Havens, Director of the Division of Research, states that "realistic limits on absorption (of aggregates) provides the most straight-forward approach to a criteria of quality from the standpoint of enforcement." The percentage of offensive aggregate, or that with high absorption, causes most concern.

The amount of wetting has considerable effect on damage caused by freezing and thawing. Damage arises wholly from the combined effects of expansion caused by freezing of absorbed water, the dilation of pressure induced, and the inherent restraining strength of the aggregate particle. Maximum damage from severe freezing occurs after the aggregate has been subjected to sustained wetting for long periods. Thus, the more water absorbed, the more severe the damage from freezing.

It is stated in the report that an oven-dried sample of aggregate will regain only about 70 per cent of its saturated moisture level in 24 hours. A number of accumulated cycles may be necessary before the sample becomes critically saturated.

The ability of an aggregate particle to withstand freezing and thawing depends on the absorption and the degree of saturation. Failures were the direct result of excessive expansion pressures. The correlations indicate that the resistance of the aggregates to freezing and thawing was dependent upon porosity, absorption, and degree of saturation. "The correlation between bulk specific gravity (saturated surface dry) and freeze-and-thaw resistance was somewhat erratic. This may have been due to the fact that some of the particles tested consisted of dolomites which are inherently heavier than limestone and siliceous gravels."

Results of tests made show that few failures occurred in particles having absorptions of less than 1 per cent. All particles having absorptions of 4 per cent or greater failed. Particles having absorptions between these extremes had increasing failures as the absorption increased. From comparisons of this with other tests results, it is obvious that absorption provides a more direct basis for judging soundness of aggregates.

As expected, particles in the higher porosity range, from 10 per cent up, fractured. Only 10 to 25 per cent of the particles having a porosity of less than 4 per cent fractured.

Igneous and metamorphic rock particles had fewer specimens which fractured than did sedimentary rock particles. "Within the sedimentary classification, limestone contained the least percentage of fractured particles, whereas dolomite contained the greatest."

The conclusions given in this paper which were drawn from test results are:

"1. The subjection of steam-saturated gravel to a quick-freeze produces dilating pressures which are damaging and can lead to failure in a single freeze-thaw cycle.

2. Particle size of aggregate tested in an unconfined state is not related to freeze and thaw durability.

3. Although most failures occur in the lower specific gravity ranges, specific gravity is not the sole indicator of aggregate durability.

4. For saturated aggregate, absorption, or its counterpart, porosity, could be used for discernment of aggregate durability.

5. Gravel particles derived from igneous and metamorphic rocks are less absorptive and more resistant to freeze-and-thaw than gravel particles derived from sedimentary rocks.

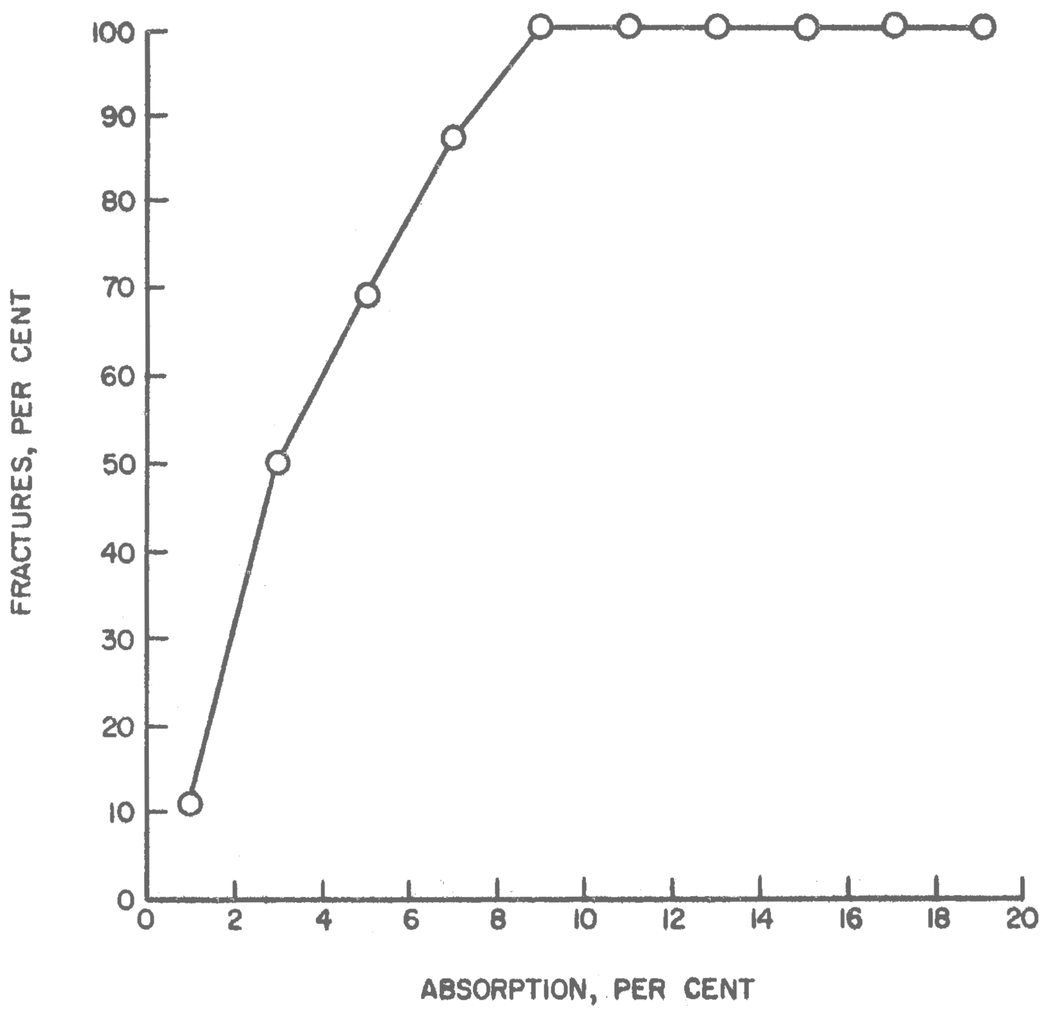
6. According to theoretical analysis, the porosity at which failure of aggregate particles can be expected occurs from 1.0 to 2.5 per cent."

(6) A. A. Anderson, a paper on Expansion Joint Practice in Highway Construction.

"This paper points out that in concrete highway pavement construction the trend is toward the elimination of expansion joints. It explains why properly maintained modern pavements do not 'blow up' even though little or no provision for expansion has been made.

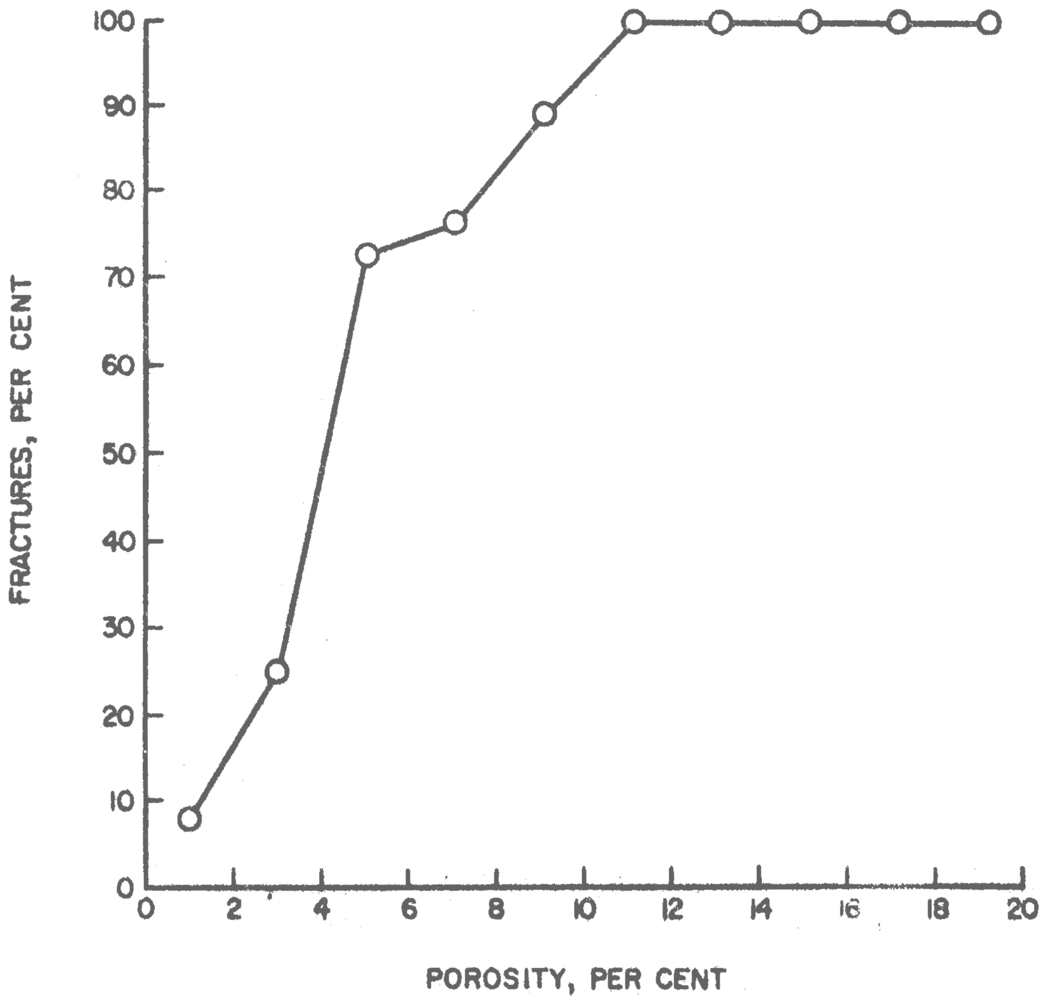
"Information taken from the reports on Michigan and Minnesota experimental jointing projects is cited to show typical slab end movements at expansion and contraction joints and how these are affected by different jointing arrangements. Using actual strain gage readings and other data from a 20-ft. section of the Minnesota experimental project where the expansion joints were 5260 ft. apart, the compression stress that developed due to an 83° F rise in the temperature of the concrete is shown to be only 628 lb. per sq. in. Based on this and the other data presented, it is concluded that in pavements built with properly spaced and maintained contraction joints, expansion joints may be eliminated, except for unusual conditions of construction."

FIGURE 1



Relationship between Absorption and Percentage of Fractured Specimens after Exposure to 4 Cycles of Freeze-and-Thaw

FIGURE 2



Relationship between Porosity and Percentage of Fractured Specimens after Exposure to 4 Cycles of Freeze-and-Thaw

(7) R.E. Davis Proceeding A.S.T.M. Vol. 30, Part I page 671, "Volume Changes in Mortars and Concrete"

Table V

Aggregate	Coefficient of Expansion per 1° F
Quartz	0.0000066
Sandstone	0.0000065
Gravel	0.0000060
Granite	0.0000053
Basalt	0.0000048
Limestone	0.0000038

The kind of aggregate also has a great effect upon volume change due to moisture variation. The table below, also by Davis, shows percentage volume change in three months.

Aggregate	Contraction in Air	Expansion in Water
Gravel	0.079	0.0074
Sandstone	0.075	0.0055
Limestone	0.039	0.0050
Granite	0.037	0.0131
Quartz	0.036	0.0094