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Contents

Marshall and Flexural Properties of Bituminous Pavement Mixtures Containing Short Asbestos Fibers

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Laboratory tests were made to evaluate the effects of the addition of short asbestos fibers on the properties of certain bituminous pavement mixtures made with typical Wisconsin aggregates. Tests included the determination of Marshall properties and of certain flexural properties, using a test method developed for the purpose. Continuous load-deflection relationships of the Marshall stability test were recorded, and the effects of a higher compactive effort on the Marshall properties were investigated.

The addition of the asbestos fibers generally improved the Marshall properties of the mixtures, and a greater range of asphalt contents could be used with less resulting loss of stability. Also, the properties of the mixtures containing the fibers tended to be affected less adversely by overcompaction. The addition of the asbestos fibers generally improved the flexural properties of the mixtures, particularly at higher temperatures.

•THE ADDITION of asbestos fibers has been shown to improve certain properties of many bituminous construction materials, and recent investigations have shown that these fibers may improve important physical properties of bituminous pavement mixtures. Promotional efforts by producers of these fibers and expressed interest by the U. S. Bureau of Public Roads suggested that an attempt should be made to evaluate the effectiveness of the fibers in improving the properties of bituminous pavement mixtures using typical Wisconsin aggregates. However, it was thought that before any large-scale pavement performance investigation be initiated, certain preliminary information should be obtained from laboratory tests. This minimal laboratory investigation was designed to yield this information.

SCOPE

This investigation included tests on three types of bituminous mixtures, each with and without the addition of short asbestos fibers in the amount of $2\frac{1}{2}$ percent by weight of the total mineral aggregate. The three mixtures were as follows:

1. A surface course mixture using a crushed gravel aggregate.
2. A surface course mixture using an aggregate composed of a blend of crushed limestone and sand.
3. A binder course mixture using the same aggregate used in mixture 2.

The tests included the determination of the standard Marshall properties of compacted specimens having varied asphalt contents and the determination of the flexural properties of compacted beam specimens. The Marshall tests were made in accordance with standard procedures, except that continuous load-deformation relationships

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TABLE 1

PROPERTIES OF AGGREGATES

Property	Crushed Gravel for Surface Course Mixture	Crushed Stone for		Wisconsin Specifications	
		Surface Course Mixture	Binder Course Mixture	Surface Course Mixture	Binder Course Mixture
Gradation, % passing sieve:					
1¼-in.					100
1-in.			100		95-100
¾-in.			93	100	-
½-in.	100	100	77	95-100	65-90
⅜-in.	90	92	67	75-100	55-80
No. 4	67	68	50	45-85	40-65
No. 10	49	49	38	30-55	25-50
No. 40	22	26	20	15-35	10-30
No. 80	10	16	12	10-25	-
No. 200	6.0	10.5	7.3	5-12	3-12
Los Angeles wear test (% loss)	21	35		50 max.	
Sodium sulfate soundness test, 5 cycles, T 104-46 (% loss)	6.0	8.5		18 max.	
Plasticity	N. P.	N. P.		<6	
Crushed particles (%)	65	-		50 min.	

were recorded throughout the loading range. The flexural test specimens were molded at optimum asphalt contents and at asphalt contents of optimum plus 2 percent, and were tested at temperatures of both 40 and 100 F.

Also, for each of the two surface-course mixtures, Marshall specimens with asphalt contents of optimum and of optimum plus 2 percent were compacted using a higher compactive effort procedure and were tested for their Marshall properties.

MATERIALS

Both aggregates were composites of several similar materials from various sources. The properties of the aggregates are given in Table 1 along with the current applicable specification requirements.

The asphalt used in these tests was an 85-100 penetration grade asphalt cement. Its properties are given in Table 2.

Asbestos fibers were furnished by the Johns-Manville Company, Asbestos Fibre Division, Manville, N. J. It was designated as their 7M06 short asbestos fiber.

TEST METHODS

Preparation of Mixtures

To insure uniformity between batches, the aggregates were separated into sev-

TABLE 2

PROPERTIES OF ASPHALT

Property	Value
Penetration, 100 g, 5 sec, 77 F	92
Specific gravity at 77 F	1.033
Flash point, C.O.C. (°F)	570
Loss on heating, 50 g, 5 hr at 325 F (%)	0.06
Penetration of residue (% of original)	87
Ductility at 77 F, 5 cm/min (cm)	110+
Solubility in CCl_4 (%)	99.9
Spot	Negative

eral size fractions and these fractions were then recombined in desired proportions for each batch. In the mixtures containing the asbestos fibers, the fibers comprised $2\frac{1}{2}$ percent by weight of the total mineral aggregate. The aggregates were heated to 270 F and the asphalt to 280 F, and these were then combined and mixed with a bowl-and-paddle type of mixer. The usual wet mixing time of 2 min was used for the nonasbestos mixtures, but this proved inadequate to assure complete and uniform mixing for the mixtures containing the asbestos fibers. For those, the heated aggregates were mixed dry for 1 min to assure uniform dispersion of the asbestos fibers and then the heated asphalt was added and the mixing continued for an additional 3 min. The compaction temperature for all test specimens was 250 ± 5 F.

Marshall Tests

The initial Marshall tests were made in accordance with standard test procedures. The test specimens were compacted with a mechanical compactor, and the voids determined using the Rice vacuum saturation procedures for obtaining the maximum void-free densities.

Continuous load-deformation measurements were recorded for one specimen representing each test condition for each of the mixtures. This was done as follows: A micrometer dial was mounted on the upper testing head and used in place of the conventional flow meter to measure flow deformation. A motion picture camera was used to record the load-deformation relationships through the complete loading range by photographing simultaneously the deformation dial and the load proving ring dial. To provide a complete picture of these relationships, the loading was continued past the indicated maximum loads and until the total deformations were about $\frac{1}{2}$ in. The load and deformation values were then read from the developed film and plotted.

Then for the two surface-course mixtures, standard Marshall test specimens were molded at both the optimum asphalt contents, as indicated by the peaks of the density curves for the nonasbestos mixtures, and at asphalt contents of optimum plus 2 percent. One series of specimens was compacted using standard compaction procedures, and another series was compacted using a considerably higher compactive effort as follows: Each end of each specimen was compacted with 50 distributed blows using the modified Proctor compaction hammer, followed by 50 additional blows using the Marshall compaction hammer. The specimens were then tested for their Marshall properties.

Flexural Tests

Flexural strength tests were made on specimens, representing each of the three types of mixtures, having optimum asphalt contents as indicated by the Marshall tests and also having asphalt contents of optimum plus 2 percent. One such series was tested at 40 F and another at 100 F.

The flexural specimens were fabricated as follows: Specimens 6 in. in diameter and about $2\frac{1}{2}$ in. in height were compacted using procedures outlined for the Hubbard-Field method of mix design (Fig. 1). Beam-type specimens 2 by 2 in. in cross-section and 6 in. in overall length were sawed from the cylindrical specimens using a diamond masonry saw. These were tested on a 5-in. span as a simple beam with center loading. The two end bearing points and the center loading point were fitted with rockers, and a 1-in. wide by $\frac{1}{8}$-in. thick steel bearing plate was used at each point to distribute the load and minimize local deformations. Figure 2 shows the testing apparatus devised for testing at 40 F. For testing at 100 F the load was applied with a motor-driven screw loading machine and loads were measured with a 60-lb capacity dynamometer. At both testing temperatures, the rate of loading was controlled at $\frac{1}{10}$ in. per minute. The testing was

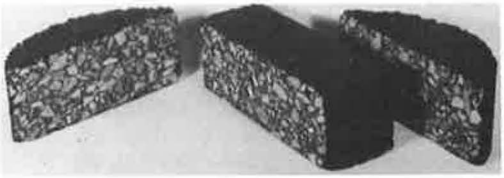


Figure 1. Sawn flexural test specimen.

done in temperature-controlled rooms, and the temperatures of the test specimens were measured with thermometers embedded in dummy specimens stored adjacent to the test specimens. In all cases the test specimens were held at the test temperatures for a minimum of 2 hr before testing. The temperatures of all specimens at the time of testing were within ± 2 F of the respective nominal test temperatures.

Load-center deflection measurements were recorded through the loading range, and these data when plotted afforded data for computing values for stiffness and modulus of rupture for each test specimen. The values for stiffness represent the slope of the initial tangent to the load-deflection curve in terms of pounds of center load per inch of center deflection, and the value for modulus of rupture were computed as:

$$\text{Modulus of Rupture (psi)} = \frac{3\,P\,l}{2\,b\,d^2}$$

in which

P = maximum center load in pounds;
l = test span length in inches;
b = specimen breadth in inches; and
d = specimen depth in inches.

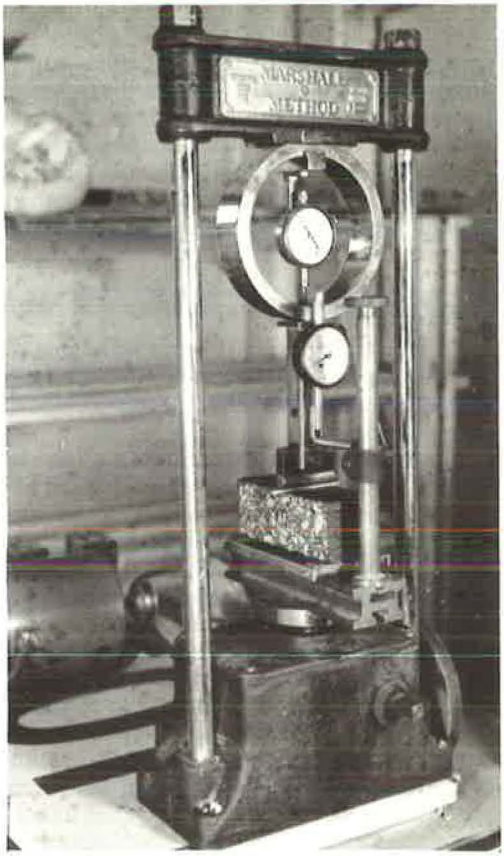


Figure 2. Low temperature flexural test apparatus.

TEST RESULTS

Marshall Design Tests

The test data are shown in Table 3 and Figure 3. For the crushed gravel surface course mixture, the addition of the asbestos fibers had no appreciable effect on the compacted density at any of the included asphalt contents but resulted in considerably higher stabilities at all asphalt contents. The addition of the asbestos fibers resulted in higher indicated void contents at all asphalt contents by amounts ranging from 0.6 to 0.8 percentage points. Flow values were essentially the same at all but the higher asphalt contents where the addition of the asbestos fibers resulted in slightly higher values.

For the crushed stone surface course mixture, the addition of the asbestos fibers resulted in lower compacted densities at all asphalt contents. Stability values were not affected greatly by the addition of the fibers in the range of likely optimum asphalt contents, but at the lower asphalt contents, the stabilities of the asbestos mixtures were slightly lower, and at the higher asphalt contents they were slightly higher than those of the corresponding nonasbestos mixtures. The voids in the asbestos mixtures were about 2.0 percentage points higher at the lowest asphalt content, and about 1.4 percentage points lower at the highest asphalt content as compared to the corresponding nonasbestos mixtures. The addition of the asbestos fibers had no appreciable effect on the flow values at any of the included asphalt contents.

For the crushed stone binder course mixture, the addition of the asbestos fibers resulted in lower compacted densities at all asphalt contents. The addition of the asbestos fibers had little effect on the indicated stabilities, except at the higher asphalt contents where the asbestos mixtures showed slightly higher stabilities than did the

TABLE 3

SUMMARY OF MARSHALL TEST DATA

Without Asbestos Fibers					With Asbestos Fibers				
Asphalt Content (% by wt.)	Bulk Density (pcf)	Stability (lb)	Flow (0.01 in.)	Voids (%)	Asphalt Content (% by wt.)	Bulk Density (pcf)	Stability (lb)	Flow (0.01 in.)	Voids (%)
(a) Crushed Gravel Surface Course Mixture									
4.75	148.6	1,153	8	4.9	4.75	148.5	1,291	7	5.6
5.5	150.0	1,235	8	3.0	5.5	150.1	1,550	8	3.6
6.25	149.9	1,375	10	2.0	6.25	150.1	1,491	8	2.6
7.0	149.2	1,122	11	1.4	7.0	149.2	1,301	14	2.2
7.75	147.9	858	14	1.2	7.75	148.0	1,081	18	2.0
(b) Crushed Stone Surface Course Mixture									
4.75	152.1	2,256	9	5.1	4.75	148.9	1,809	8	7.1
5.5	152.6	2,109	9	3.9	5.5	151.0	2,028	11	4.5
6.25	152.2	1,655	12	3.3	6.25	151.4	1,841	12	3.1
7.00	151.4	1,248	14	3.0	7.0	150.8	1,462	15	2.2
7.75	149.7	889	21	3.2	7.75	149.4	1,195	19	1.8
(c) Crushed Stone Binder Course Mixture									
4.0	152.2	2,136	8	6.1	4.0	149.4	2,086	12	8.6
4.75	153.5	2,085	10	4.1	4.75	151.3	2,024	10	6.2
5.5	153.7	1,768	11	2.7	5.5	151.8	1,864	12	4.7
6.25	152.7	1,175	16	2.1	6.25	151.6	1,479	16	3.5
7.00	151.2	929	23	1.7	7.00	150.7	1,140	22	2.8
					7.75	149.4	954	33	2.2

corresponding nonasbestos mixtures. The voids in the compacted asbestos mixtures were higher at all asphalt contents than those of the corresponding nonasbestos mixtures by amounts ranging from 1.1 to 2.5 percentage points. The addition of the asbestos fibers had no appreciable effect on the indicated flow values at any of the included asphalt contents.

The Marshall test load-deformation relationships are shown in Figure 4. The peaks of the curves represent the stability values, and the deformations at these peaks represent the flow values. For the mixtures containing the asbestos fibers, the loads, after reaching their maximums, did not drop off as greatly or as rapidly as did those for the corresponding nonasbestos mixtures, but continued to carry an appreciable part of the maximum loads even after total deformations of up to about ½ in. This was true for each of the three mixtures at all of the included asphalt contents.

Effects of High Compactive Effort on Marshall Properties

The test data for the effects of high compactive effort on Marshall properties are shown in Table 4 and Figure 5.

Crushed Gravel Surface Course Mixtures.—At optimum asphalt content, the higher compactive effort resulted in greater densities for both the nonasbestos and the asbestos mixtures, the increase for the asbestos mixtures being considerably less than for the nonasbestos mixtures. At the higher asphalt content, the higher compactive effort resulted in higher density for the nonasbestos mixture but in slightly lower density for the mixture containing asbestos. This apparent anomaly may be related to the observed behavior of the asbestos mixture during compaction. The mixture became quite elastic and showed considerable rebound, and the top surfaces of the specimens after compaction showed considerable convexity.

At optimum asphalt content, the higher compactive effort resulted in considerable increase in stability for the nonasbestos mixture, but caused very little change in the stability of the asbestos mixture. However, the asbestos mixtures compacted by either procedure showed higher stabilities than did the corresponding nonasbestos mixtures. At the higher asphalt content, the higher compactive effort resulted in slightly higher stability for the nonasbestos mixture but in slightly lower stability for the asbestos mixture. Again the asbestos mixtures compacted by either compaction procedure showed higher stabilities than did the corresponding nonasbestos mixtures.

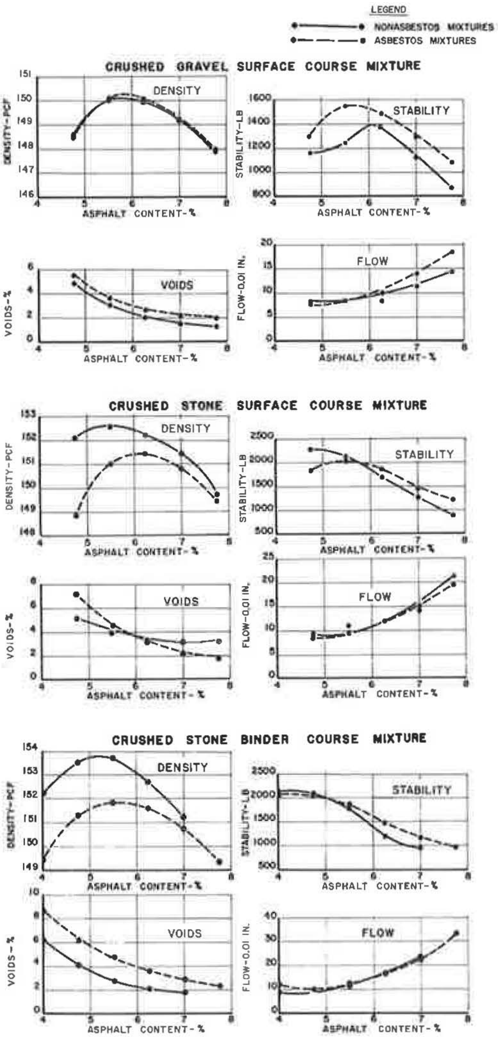


Figure 3. Marshall test data.

At optimum asphalt content, the higher compactive effort resulted in considerable reduction in void content for both the non-asbestos and asbestos mixtures, the reduction for the asbestos mixtures being somewhat less. Also, for either compactive effort the voids in the asbestos mixtures were greater than those in the corresponding nonasbestos mixtures. At the higher asphalt content, the higher compactive effort reduced the voids in the non-asbestos mixture to about 0.4 percent which may be a dangerously low value, whereas for the asbestos mixtures, the higher compactive effort actually resulted in slightly higher void content, the possible reason for which was discussed when considering the densities of these mixtures.

Flow values generally increased with the increased asphalt content, greater compactive effort, and addition of the asbestos fibers.

Crushed Stone Surface Course Mixtures. —At optimum asphalt content, the higher compactive effort resulted in greater densities for both nonasbestos and asbestos mixtures, the increase for the asbestos mixtures being considerably less than that for the nonasbestos mixtures. At the higher asphalt content, the higher compactive effort resulted in slightly higher densities for both nonasbestos and asbestos mixtures.

At optimum asphalt content, the higher compactive effort resulted in slightly reduced stability for the nonasbestos mixture and in slightly increased stability for the asbestos mixture. At the higher asphalt content, the higher compactive effort resulted in very little change in the stabilities of either the nonasbestos or the asbestos mixture. The asbestos mixtures under either compaction procedure with either asphalt content showed higher stabilities than did the corresponding nonasbestos mixtures.

At optimum asphalt content, the higher compactive effort resulted in lower void contents for both the nonasbestos and the asbestos mixtures, the reduction being slightly less for the asbestos mixture. At the higher asphalt content, the higher compactive effort resulted in slightly lower void contents for both the nonasbestos and the asbestos mixtures. Also, for either compactive effort, the asbestos mixtures had lower void contents than did the corresponding nonasbestos mixtures.

Flow values increased with increased asphalt content and with greater compactive effort, but generally the asbestos mixtures had lower flow values than did the corresponding nonasbestos mixtures.

Flexural Properties of Beams

The load-deflection data from the flexural tests are shown in Figure 6. Table 5 gives the properties of the test specimens including values for stiffness and modulus

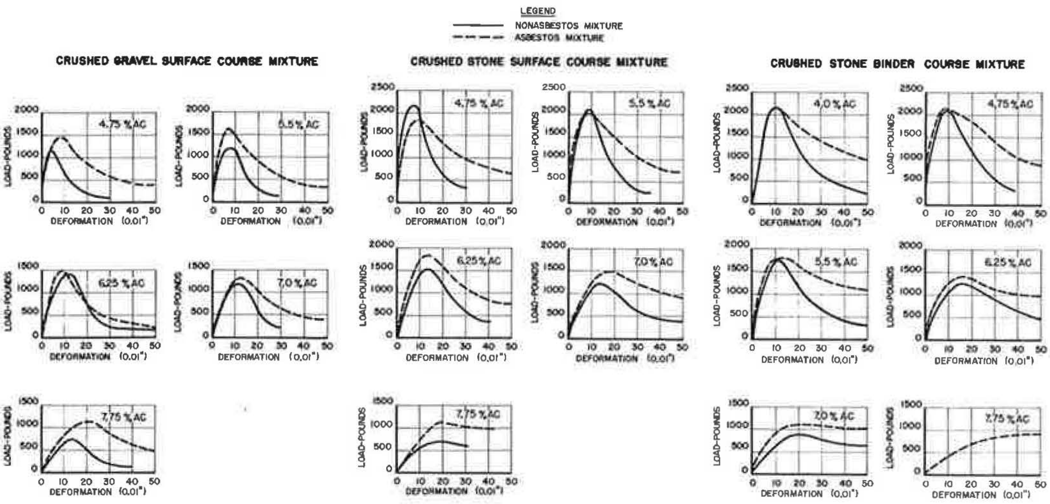


Figure 4. Marshall load-deformation relationships.

TABLE 4

EFFECTS OF HIGH COMPACTIVE EFFORT ON MARSHALL PROPERTIES

Asphalt Content (% by wt.)	Degree of Compaction	Properties of Compacted Mixtures: Without Asbestos Fibers: Bulk Density (pcf)	Stability (lb)	Flow (0.01 in.)	Voids (%)	With Asbestos Fibers: Bulk Density (pcf)	Stability (lb)	Flow (0.01 in.)	Voids (%)
(a) Crushed Gravel Surface Course Mixture									
5.7	Standard	149.6	1,120	7	3.0	150.3	1,640	8	3.2
	High	152.3	1,410	10	1.3	152.1	1,675	13	2.0
7.7	Standard	147.4	625	15	1.6	148.1	995	16	2.1
	High	149.3	715	22	0.4	147.3	855	31	2.5
(b) Crushed Stone Surface Course Mixture									
5.5	Standard	152.4	2,005	9	4.1	152.0	2,030	9	4.0
	High	154.8	1,830	18	2.6	153.6	2,365	15	2.9
7.5	Standard	150.0	875	23	3.4	149.6	1,105	20	2.1
	High	150.3	835	35	3.2	149.8	1,145	31	2.0

of rupture obtained from the load-deflection curves. The latter data are shown in Figures 7 and 8 for easier comparisons. In Figure 7, when tested at 40 F those mixtures containing the asbestos fibers showed higher moduli of rupture as compared to those of the corresponding nonasbestos mixtures, except in the case of the crushed stone surface course mixture at optimum asphalt content and the crushed stone binder course mixture at optimum asphalt content. When tested at 100 F, the mixtures containing asbestos fibers had higher moduli of rupture in all cases as compared to the nonasbestos mixtures, though the difference is quite small in the case of the crushed stone surface course mixture having optimum asphalt content.

In Figure 8, the stiffness values of the mixtures tested at 40 F containing asbestos fibers were equal to or greater than those of the corresponding nonasbestos mixtures in all cases, the greatest difference being for those mixtures having the higher than optimum asphalt contents. When tested at 100 F, the stiffness in all cases was greatly increased by the addition of the asbestos fibers.

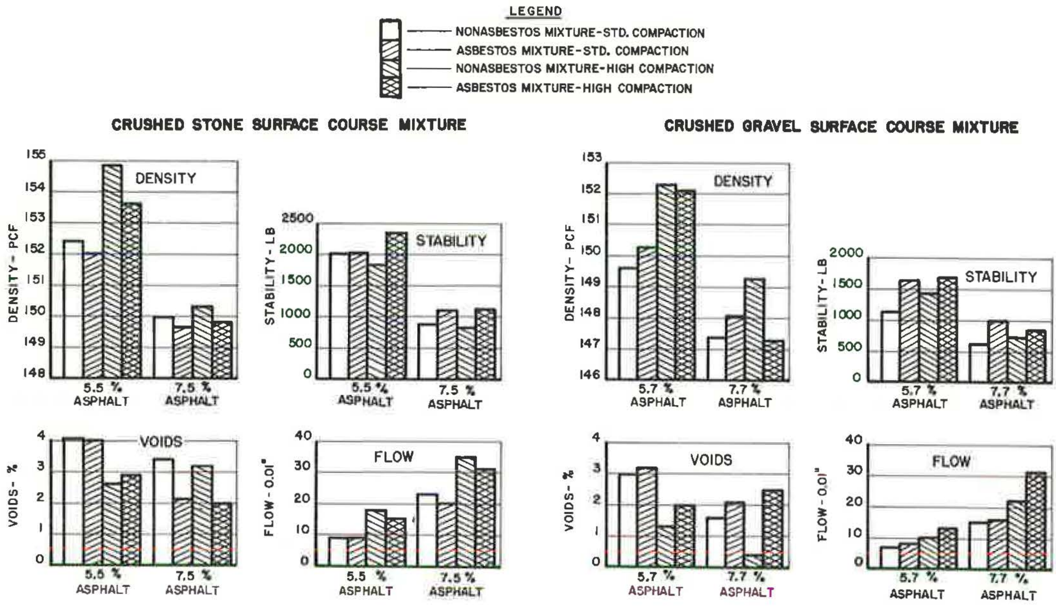


Figure 5. Effects of high compactive effort on Marshall properties.

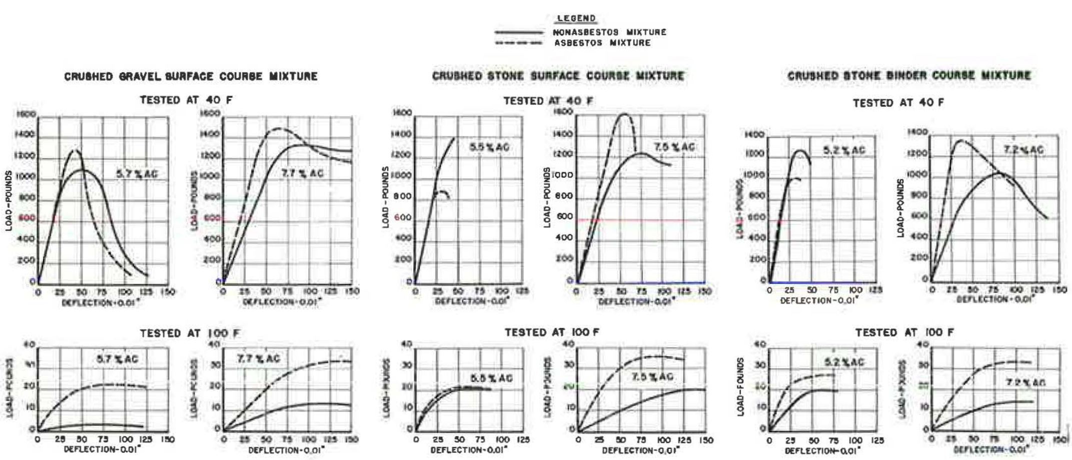


Figure 6. Flexural load-deflection relationships.

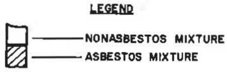
ANALYSIS OF TEST RESULTS

In the design of bituminous paving mixtures, the primary properties considered include stability and durability. Stability or resistance to plastic deformation, particularly at high temperatures, is necessary to preclude rutting, shoving, or other forms of pavement displacement, and durability is important in maintaining the structural integrity and surface characteristics of the pavement under exposure to weather and traffic. Flexibility and fatigue resistance are also important, particularly at lower temperatures, if the pavement is to conform to variations in base elevations, and to flex repeatedly under traffic without cracking. It is generally recognized that durability, flexibility, and fatigue resistance improve with increases in asphalt content, but it is also recognized that increased asphalt content may result in lowered stability.

TABLE 5

SUMMARY OF FLEXURAL TEST DATA

Test Temperature (°F)	Asphalt Content (% by wt.)	Without Asbestos Fibers: Bulk Density (pcf)	Voids (%)	Maximum Center Load (lb)	Stiffness (lb/in.)	Modulus of Rupture (psi)	With Asbestos Fibers: Bulk Density (pcf)	Voids (%)	Maximum Center Load (lb)	Stiffness (lb/in.)	Modulus of Rupture (psi)
(a) Crushed Gravel Surface Course Mixture											
40	5.7	146.1	5.3	1,100	3,280	1,030	145.0	6.5	1,280	3,600	1,200
	7.7	146.5	2.2	1,320	2,120	1,240	147.5	2.4	1,490	3,120	1,400
100	5.7	146.3	5.2	4	10	4	145.6	6.2	23	76	22
	7.7	145.3	3.0	14	18	13	147.5	2.4	34	40	32
(b) Crushed Stone Surface Course Mixture											
40	5.5	148.8	6.3	1,400	4,000	1,000	144.1	8.9	880	4,000	820
	7.5	149.0	4.0	1,250	2,600	1,170	148.9	2.6	1,620	3,400	1,520
100	5.5	148.5	6.5	21	96	20	144.5	8.7	22	120	21
	7.5	149.3	3.8	20	20	19	149.5	2.2	36	88	34
(c) Crushed Stone Binder Course Mixture											
40	5.2	149.9	5.6	1,280	4,400	1,200	146.2	8.7	1,000	4,960	940
	7.2	150.1	2.1	1,040	2,200	970	150.0	2.8	1,350	4,960	1,270
100	5.2	150.6	5.2	20	48	19	146.9	8.2	28	104	26
	7.2	150.5	1.8	14	20	13	150.1	2.8	34	80	32



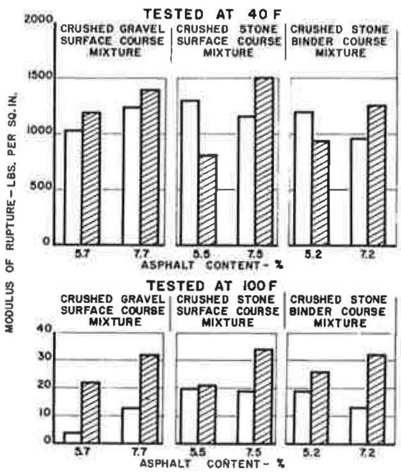


Figure 7. Modulus of rupture of beam specimens.

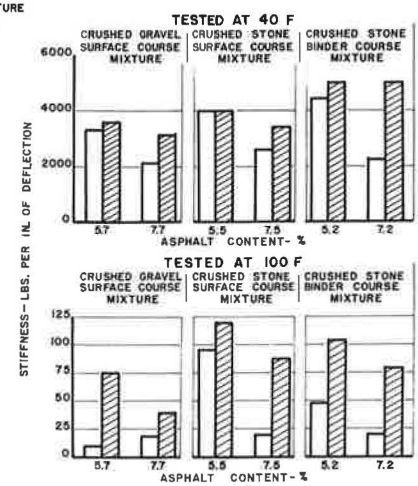


Figure 8. Stiffness of beam specimens.

Stability or Resistance to Plastic Deformation

The Marshall design test data indicate that for the crushed gravel surface course mixture, the addition of the asbestos fibers resulted in increased stabilities at all included asphalt contents, and that the asphalt content could be increased appreciably without loss of stability as compared to that of the nonasbestos mixture having optimum asphalt content. For the crushed stone surface course mixture and for the crushed stone binder course mixture, the addition of the asbestos fibers had little effect on the Marshall stabilities in the range of likely optimum asphalt content, for both mixtures, increases in asphalt content above the optimum resulted in less reduction in stability for the asbestos mixture.

The load-deformation data for the Marshall test specimens indicate that for each of the three mixtures at all included asphalt contents, the addition of the asbestos fibers resulted in greater retained load-carrying ability of the test specimens after the indicated maximum loads. Although the importance of this increased load-carrying ability of the test specimens may not be readily evaluated, it does reflect an increase in resistance to plastic deformation which possibly could contribute to improved pavement performance.

Also, for the two surface course mixtures at asphalt contents of both optimum and optimum plus 2 percent, when compacted with either standard or high compactive effort, those mixtures containing the asbestos fibers showed higher Marshall stabilities than the corresponding nonasbestos mixtures did.

The flexural data for beam-type specimens tested at 100 F indicate that for all three mixtures, at asphalt contents of either optimum or optimum plus 2 percent, the moduli of rupture of the asbestos mixtures were greater than those of the corresponding nonasbestos mixtures by amounts ranging from 5 to 450 percent, and that the flexural stiffness values for the asbestos mixtures were greater than those of the corresponding nonasbestos mixtures by amounts ranging from 25 to 660 percent.

Flexibility at Low Temperatures

The flexural test data for the specimens tested at 40 F indicate that the addition of the asbestos fibers resulted in increased stiffness for all three mixtures having asphalt contents of either optimum or optimum plus 2 percent, except in the case of the crushed stone surface course mixture having optimum asphalt content where the addition of the asbestos fibers caused no change in stiffness. Although increased stiffness indicated in these tests may be considered an indication of lowered flexibility, for the two surface-course mixtures, the asbestos mixtures having higher than optimum asphalt contents had lower indicated stiffness values than did the corresponding nonasbestos mixtures having optimum asphalt contents. Also, the mixtures containing the asbestos fibers had higher moduli of rupture at asphalt contents of either optimum or optimum plus 2 percent, as compared to the nonasbestos mixtures except for the crushed stone surface course mixture at optimum asphalt content, or for the crushed stone binder course mixture at optimum asphalt content.

CONCLUSIONS

This investigation, though designed to yield certain basic information regarding the effects of the addition of asbestos fibers on certain of the properties of bituminous paving mixtures, is not broad enough in scope to cover all the variables existing in bituminous pavement construction. Also, it is fully recognized that small-scale laboratory test data cannot be reliably extrapolated to predict the behavior of pavements under field conditions. However, within the scope of this investigation and under the conditions of testing and evaluation, it is indicated that the addition of the asbestos fibers to bituminous pavements mixtures will generally improve the Marshall properties of the mixtures and allow the use of a greater range of asphalt contents with less resulting loss of stability and thus contribute to improved durability, flexibility, and fatigue resistance.

It is also indicated that the Marshall properties of mixtures containing the asbestos fibers tended to be affected less adversely by overcompaction.

The flexural test data indicate that the addition of the asbestos fibers generally improved the flexural properties of the mixtures, particularly at higher temperatures.

Certain of the data would indicate that bituminous mixtures containing crushed gravel aggregates may be benefited more by the addition of the asbestos fibers than would similar mixtures containing crushed stone aggregates.

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2. "Mix Design Methods for Hot-Mix Asphalt Paving." Asphalt Institute Manual Series 2.

3. Monosmith, C. L., "Flexibility Characteristics of Asphaltic Paving Mixtures." Proc., AAPT, Vol. 27 (1958).
4. Kietzman, J. H., "Effect of Short Asbestos Fibers on Basic Physical Properties of Asphalt Pavement Mixes." HRB Bull. 270, 1-19 (1960).
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Performance of Asbestos-Asphalt Pavement Surface Courses with High Asphalt Contents

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The performance of eight asbestos-asphalt pavements and adjacent standard pavements placed in 1959 and 1960 in four cities is evaluated through core analyses, surface texture photographs, water permeability, and skid resistance tests.

Even with asphalt content increased 50 percent or more above standard optimum, the pavements containing asbestos fibers have remained stable under heavy traffic. Asphalt hardening has been greatly reduced, with penetration of recovered asphalt at two locations remaining within the range of original penetration grade. All these test pavements with high asphalt contents show superior resistance to incipient raveling compared with adjacent standard pavements. Simple mix design criteria have been established for asbestos-asphalt surface pavements based on superior performance of these test sections to date.

•FOUR YEARS AGO an investigation of asbestos-asphalt pavements was initiated by Johns-Manville Research and Engineering Center to study the effect of short asbestos fibers on the various physical properties of asphalt pavement. The type of asbestos used was Canadian chrysotile asbestos, which makes up more than 90 percent of the fiber used in the United States for all purposes. It is a nonproprietary product with at least nine producers in Canada. The fiber grade chosen for the evaluation was 7M06, a fiber used in a variety of asphalt building and industrial products for many years. Tables 1 and 2 give the asbestos fiber classification and physical properties of Canadian chrysotile fiber.

Results (1) of the wide variety of laboratory tests performed in the study were subsequently presented at the 1960 Highway Research Board annual meeting. Included in the early work were static load compression tests, which suggested that with asbestos included, asphalt could be increased above standard optimum by 40 to 50 percent. Based on the acknowledged fact that an asphalt pavement was weaker in static loading compared to dynamic loading, these test data were used as a basis for mix design recommendations for asphalt concrete with asphalt content increased from approximately 6 to 7.5 or 8 percent. In 1959 and early 1960, a number of cities placed test pavements following these recommendations, and in some instances increased asphalt content up to 9 percent or more by weight of total mix.

The following is a progress report on the performance of these early test pavements with high asphalt content compared to the adjacent standard pavements placed simultaneously. The performance data include photographs of the present pavement surfaces, core analyses, stability test data, and permeability and skid resistance test results. Tentative interpretations are made of the performance data and test results in light of past performance studies of asphalt pavements published by the industry. Only that phase of performance which relates to durability of the asphalt surface course

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is considered in this report. Excluded is the other important phase of performance which relates to structural design. The report includes all the test pavements with high asphalt contents placed before June 1960. In addition, the core analyses represent all the cores taken from these pavements to date.

For some time after June 1960, Johns-Manville paving recommendations deliberately limited the increase in asphalt content in order to study field performance of these high asphalt content pavements and thereby confirm the laboratory test data. During this "performance period," tests by the American Oil Company (2) using a full-scale traffic simulator demonstrated that under very heavy traffic, asphalt content in asbestos-asphalt concrete could be increased safely up to 50 percent above the standard optimum value. These tests supported the field performance results; in addition, they established quantitatively the relationship between fiber-to-asphalt ratio and critical temperature resistance.

Review of Literature

Before initiation of laboratory work on asbestos-asphalt pavements in 1959, a number of leading asphalt paving technologists were consulted to determine in what way they thought asphalt pavements could best be improved in performance.

There was general agreement that increasing asphalt content or film thickness of the binder between aggregate particles should improve pavement durability. Published reports confirmed this conclusion.

Reference to a few of these reports is pertinent and will establish a frame of reference for the report to follow.

In a performance study by Raschig and Doyle (3) in 1937, a qualitative correlation was found between penetration of recovered asphalt and condition of the pavement surface (i.e., raveling and cracking) as shown in Table 3.

In 1937, Hubbard and Gollomb (4), in their study of 29 pavements, reported the same general relationship with the critical range given as 20 to 30 penetration. Their interest in this was prompted by a survey in Ohio which showed that roads built with 50-60 grade asphalt went from good to poor quality in the age interval between 38 and 53 months, corresponding to a penetration of 32 and 25, respectively.

TABLE 1

ASBESTOS FIBER CLASSIFICATION

Quebec Standard Classification		Quebec Standard Test[a] Guaranteed Minimum Wt. (oz)			
Group	Item	½ In.	No. 4	No. 10	Pan
3	3R	2	8	4	2
4	4K	0	4	9	3
5	5R	0	0	10	6
6	6D	0	0	7	9
7	7D	0	0	5	11
	7F	0	0	4	12
	7H	0	0	3	13
	7K	0	0	2	14
	7M	0	0	1	15
	7R	0	0	0	16

[a]Canadian chrysotile asbestos classification.

TABLE 2

PHYSICAL PROPERTIES OF CANADIAN CHRYSOTILE ASBESTOS

Property	Value
Specific gravity[1]	2.55
Fiber diameter[2] (in.)	0.000000706 to 0.00000118
No. of fibrils[1] in 1 in.	850,000 to 1,400,000
Tensile strength[1,2] (psi)	100,000 to 355,000

[1]Source: (15).
[2]Source: (16).

TABLE 3

CONDITION OF PAVEMENT SURFACE AND PENETRATION OF RECOVERED ASPHALT

No. of Pavements	Condition	Penetration[a]
8	Excellent	40
4	Good	29
5	Fair	20
12	Poor	14

[a]At 77 F recovered asphalt.

As a result, softer asphalts were specified for general use by Ohio, California, and, subsequently, most States during the 1940's. In 1939, Vokac (5) confirmed this but reported the critical range to be between 18 and 25 penetration.

One of the most comprehensive studies of durability of road asphalts was published in 1942 by Endersby, Stross, and Miles (6). In summarizing the work of others they reported:

> This experience is paralleled by that in Europe where it has been found that pavements tend to crack in the region of 20 penetration but are in good condition above 30...
>
> There is a climatic factor, since in Arizona bad results do not seem to follow until penetration drops below 15...Mere hardness cannot in itself be the sole cause of raveling and cracking ...However, the tendency of an asphalt to harden is one of the indispensable factors of any predictive formula, because the deleterious changes, whatever they may be, are always accompanied by hardening.

Endersby and his co-writers, in setting up a rating system, relied on "raveling resistance . . . as an index of durability" because "cracking is very much a function of the resilience of the subgrade and its resistance to depression under load." Applying their numerical rating system from 1 to 6 (with 1 being top rating) they found that, "raising asphalt content in this mix by one-fifth raises the grade 1 point, except of course, where the rating was already 1."

Recently, a report (7) prepared jointly by the U. S. Bureau of Public Roads Laboratory and the Maryland State Roads Commission was published describing the performance of wearing courses containing 85-100 penetration asphalt. The relationship found between initial air voids and 4-year loss in penetration is shown in Figure 1. Because of the rapid loss in penetration in pavements with high initial void contents, they

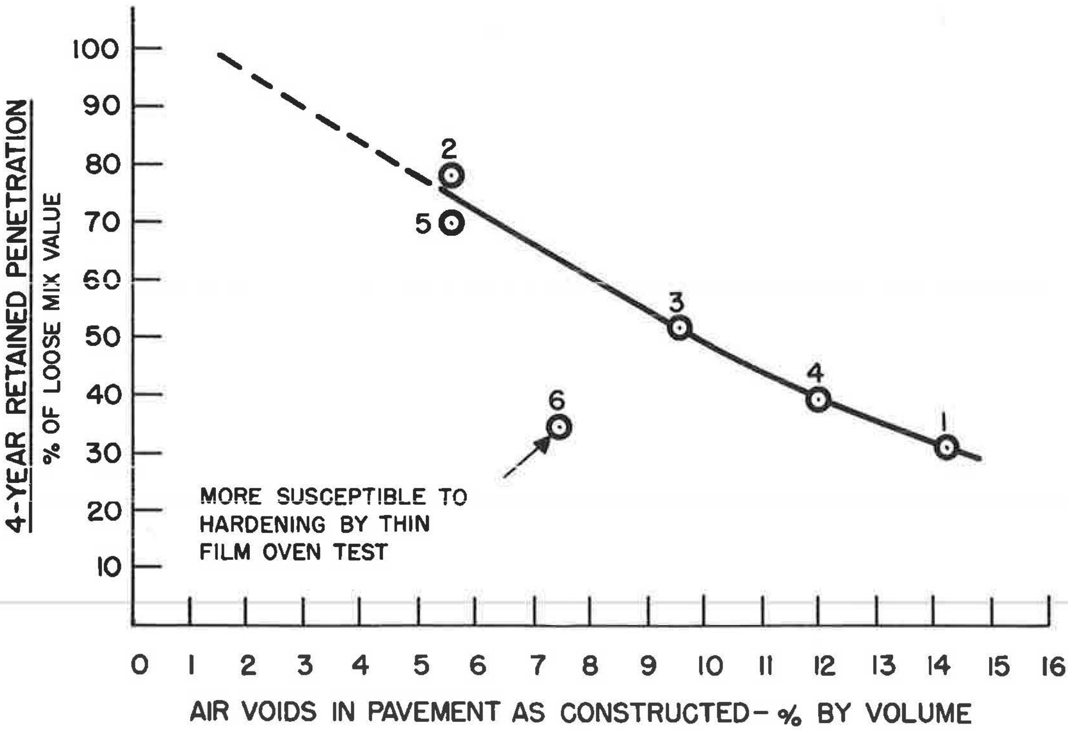

Figure 1. Effect of initial percentage of air voids in pavement (from 7).

recommend limiting initial void contents for heavy-duty pavements to approximately 7 percent maximum. The authors point out that "percentage of air voids is controlled primarily by asphalt content for a particular aggregate," and therefore, asphalt content could be adjusted in the field so as to obtain initial voids after compaction within prescribed limits. They also note that "asphalt ductility did not drop below 150 cm for the three test sections. . . with the lowest percentage of air voids." In the pavements with high initial voids after compaction under traffic, ductility is shown to be decreasing rapidly, whereas corresponding penetration is decreasing slowly. These data suggest that both ductility and penetration measurements are desirable in studying hardening of asphalt in pavement as related to performance.

Some Factors Affecting Durability

It is not practical to review all the published work on durability as it relates specifically to bituminous surfacing. Basically, apart from structural performance, durability refers to (a) possible water susceptibility characteristics related to the mineral constituents, and (b) hardening of the asphalt binder. Data published by the Asphalt Institute indicate that in contrast to many fine mineral admixtures, asbestos has no detrimental effect on water susceptibility characteristics (8, 9). Unpublished results of many other laboratories confirm this fact.

Concerning hardening of asphalt, several test series have been performed on asbestos-asphalt mixes by the Chicago Testing Laboratory for Johns-Manville using the Shattuck oxidation test to measure high-temperature hardening during mixing, and aging tests to measure hardening under service conditions. The report on the Shattuck oxidation tests conducted on each of six asphalts produced from different crude oils mixed with 2.5 percent 7M06 chrysotile fiber and Ottawa silica sand concluded:

> The asbestos has no adverse effect upon the hardening of the asphalt during the mixing cycle...Asbestos can safely be used in bituminous paving mixtures without any adverse effects upon the asphalt cement. (Appendix A)

The report on "The Effect of Aging Upon the Properties of Asphalt Concrete Containing Asbestos" concluded:

> Based upon these test results, it is apparent that asbestos has no injurious effect upon the properties of the asphalt in hot mix bituminous concrete when subjected to one year of aging. (Appendix B)

TEST METHODS

Table 4 gives basic information to identify the various pavements described. Figures 2 through 5 show the exact locations of each of the mixes including the approximate location of the cores described in this report. Aggregate gradations are shown in Figure 6. Production and construction information are given in Table 5. Core and plant mix analyses made at the time of placement are given in Appendixes C and D and recent core analysis data are given in Tables 6 through 9.

The methods used by the Chicago Testing Laboratory are outlined as follows:

1. Bulk density -- ASTM test method D-1188.
2. Theoretical maximum specific gravity — Michigan solvent immersion method.
3. Asphalt extraction (bitumen content) — ASTM D-1097.
4. Asphalt recovery — Abson method (ASTM D-1856 - 61T).

The surface layers in the cores were cut from the base or binder layers so that a small amount of the surface mix remained with the binder or base layer to prevent "contamination" in the analyses.

Skid resistance tests on the Manville pavements in October 1960 employed the stopping distance method with a Wagner Stopmeter and fifth wheel. The tests were performed on wet pavements at 30 mph, using a standard passenger car. Details of the apparatus and test conditions are given in Appendix E.

TABLE 4

PAVEMENT IDENTIFICATION

Location		Mix No.[a]	Date Placed	Placed by	Estimated Traffic	
City	Street				Count per Day	% Heavy Trucks
Calgary	Alyth Freeway	1 2	June 1960	City Street Dept.	6,500	10 (est)
Dallas	Greenville		Nov. 1959	M. P. McInerney Co.	23,000	
	Ross Avenue	1 2			17,000	15 (est)
Manville	N. 13th	1 2	Sept. 1959	Jannarone Engineering Company	500 (est)	10 (est)
St. Louis	Manchester	1	Oct. 1959	Bridges Paving Co.	10,300	25
	(Tower Grove) Strodtman	2 3			500-1,000 (est)	Min. 10 (est)

[a]Identifies exact mix described in Appendixes.

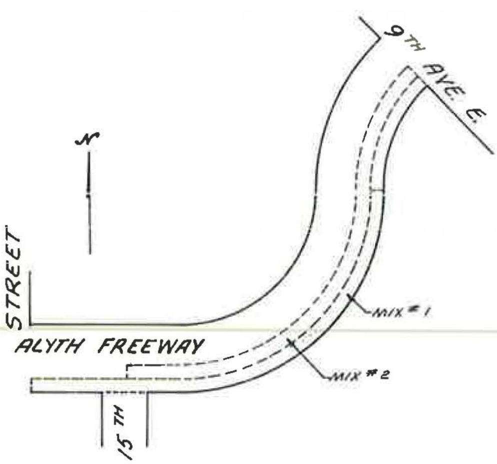

Figure 2. Mix locations, Calgary, Alberta.

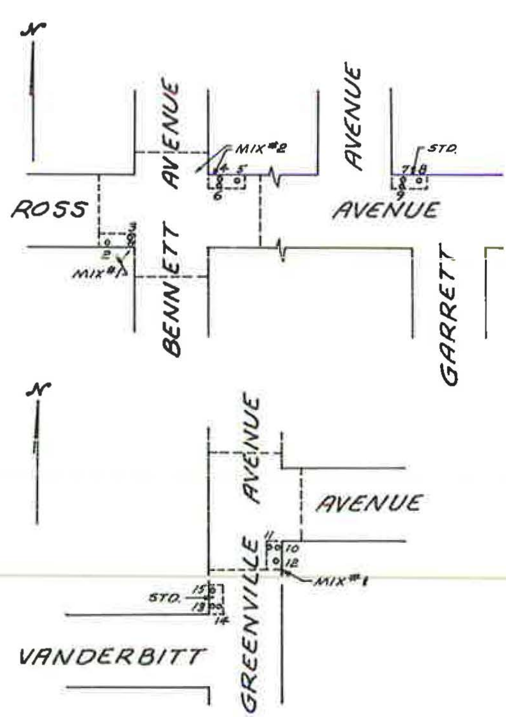

Figure 3. Mix locations, Dallas, Texas.

TABLE 5

PRODUCTION AND CONSTRUCTION DATA OF ASBESTOS SECTIONS

Location	Mix No.	Mixing Temp.[1] (°F)	Mixing Time (sec) Dry	Mixing Time (sec) Wet	Paving Machine	Compaction Method	Placement Characteristics	Length (ft)	Average Width (ft)	Average Thickness (in.) Design	Average Thickness (in.) Core	Type of Pavement	Type of Subbase Mix (State spec.)	Binder	Base, Subbase Underlying Pavement
Calgary	1		15	45	Barber-Greene	Steel wheel tandem	Flushed under roller	1,260	10	1½	1.58	New construction	HL III	1½-in. standard	2 in. of ¾-in. gravel
	2		15	45	Barber-Greene		No flushing under roller, good placeability			1½	1.46			(without asbestos)	17 in. of pit run gravel
								1,082	10		1.62 (std)				
Dallas	1	275-350	8	52	Blaw-Knox	Steel wheel roller-10 tons (3-wheel gallion Roll-O-Matic)	Good	90	46	2	1.33	Overlay	Type B	5-in. Type C	Asphalt
	2		8	52	Blaw-Knox		Good	125	65	2	1.16			(without asbestos)	Concrete
											1.06 (std)				
Manville	1	300-325	60	wet mix	Barber-Greene	Steel wheel-tons	Slight flushing under roller[2]	271	20	1-1¼	1.32	New construction	SM	None	Penetration
	2	300-325	60	wet mix	Barber-Greene	Steel wheel-tons	Considerable flushing under roller[2]	242	20	1-1¼	1.35	New construction	SM	None	Macadam
											1.05 (std)				
St. Louis	1	325			Barber-Greene	Steel wheel roller—tons	Some flushing under roller	250	11	1½		Overlay		None	Paving brick
	2	325			--	Steel wheel roller—tons	Some flushing under roller	125	11	1½		Overlay		None	Paving brick
	3	325			--	Steel wheel roller—tons	Some flushing under roller, good placeability								

[1]Same mixing temperature used in both standard and asbestos pavements at each location.
[2]Mix as well as standard showed considerable tearing of mat; rolling eliminated tear cracks.

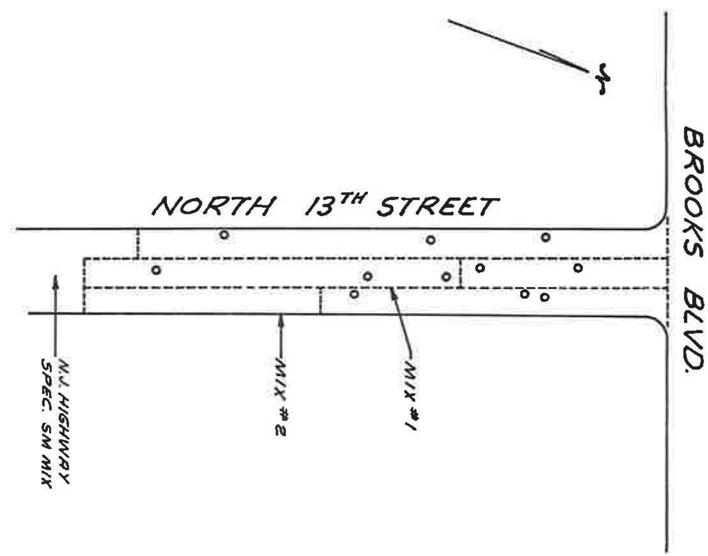

Figure 4. Mix Locations, Manville, N. J.

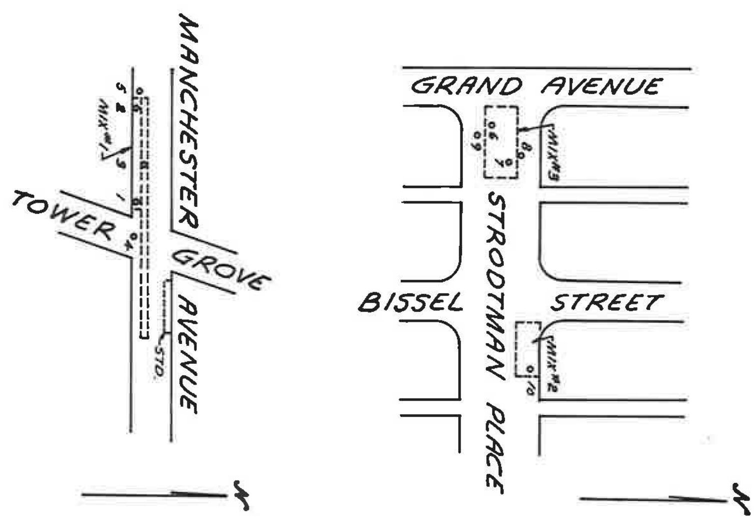

Figure 5. Mix Locations, St. Louis, Mo.

TABLE 6

CALGARY, ALBERTA, CORE ANALYSES, CTL REPORTS 11537 AND 10364-69

Location (set)	Core No.	Specific Gravity at 77 F	Theoretical Maximum Specific Gravity	Air Voids (%)	Surface Course Thick. (in.)	Designed Composition		Recovered Bitumen (%)	Recovered Asphalt		
						Asbestos	Asphalt		Pen. at 77 F, 100/5	Duct. at 77 F, 5/60 cm	Ash (%)
1	10364	2.34	2.39	2.1	1.50						
	10365	2.35	2.39	1.7	1.63	3-7M	6.9-9.2	8.0	111	150+	1.9
	10366	2.34	2.39	2.1	2.00						
	10367	2.37	2.48	4.5	1.75						
	10368	2.33	2.48	6.0	1.63	-	-	5.3	51	150+	1.7
	10369	2.39	2.48	3.6	1.63						
2	1	2.36	2.48	4.8	2.13	0	5.0	4.7	48	150+	1.1
	2	2.41	2.46	2.0	1.38	0	5.0	4.8	63	150+	1.6
	3	2.35	2.38	1.3	1.25	3-7M	7.0	7.4	78	150+	0.6
	4	2.41	2.46	2.0	1.38	0	5.0	5.0	58	150+	1.0
	5	2.35	2.38	1.3	1.63	3-7M	7.0	7.0	83	150+	1.0
	6	2.32	2.38	2.5	1.50	3-7M	7.0	7.4	79	150+	0.3
	7	2.35	2.39	1.7	1.50	3-7M	7.0	7.1	67	150+	1.3
	8	2.34	2.39	2.1	1.50	3-7M	7.0	7.2	65	150+	0.9
	9	2.34	2.38	1.7	1.38	3-7M	7.0	7.1	62	150+	1.2

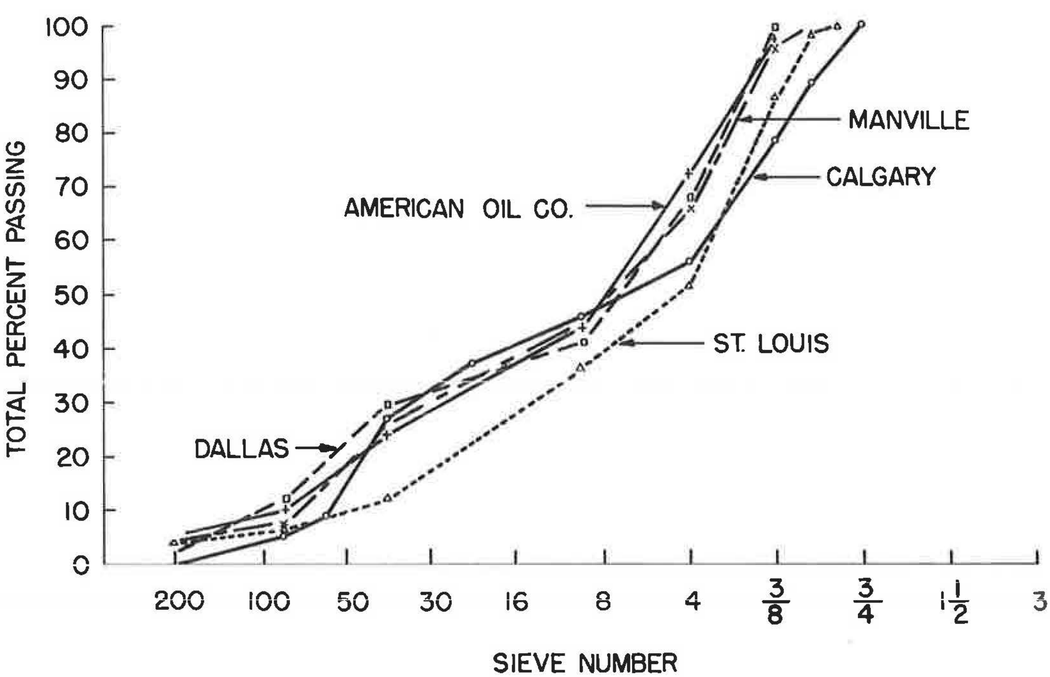

Figure 6. Aggregate gradation from core analyses.

TABLE 7

DALLAS, TEXAS, CORE ANALYSES, CTL REPORTS 04406-11 AND 11428

Location	Core No.	Specific Gravity at 77 F	Theoretical Maximum Specific Gravity	Air Voids (%)	Surface Course Thick. (in.)	Designed Composition		Recovered Bitumen (%)	Recovered Asphalt		
						Asbestos	Asphalt		Pen. at 77 F, 100/5	Duct. at 77 F, 5/60 cm	Ash (%)
Ross	1	2.35	2.38	1.2	1.25						
Ave.	2	2.28	2.38	4.2	1.13	1.7-7M	7.5	6.5	40	93	1.2
(1962)	3	2.32	2.38	2.5	1.62						
	4	2.33	2.37	1.7	1.25						
	5	2.33	2.37	1.7	1.25	2.6-7M	7.5	6.9	40	98	1.7
	6	2.33	2.37	1.7	1.00						
	7	2.32	2.46	5.7	1.25						
	8	2.33	2.46	5.3	1.25	0	5.0	5.0	25	12	1.7
	9	2.33	2.46	5.3	1.25						
Ross	1	2.36	-	-	-	2.6-7M	7.5	6.9	37	49	1.5
Ave.	2	2.29	-	-	-	2.6-7M	7.5	6.8			
(1961)	3	2.36	-	-	-	2.6-7M	7.5	7.3	38	45	2.5
	4	2.32	-	-	-	2.6-7M	7.5	-	-	-	-
	5	2.36	-	-	-	2.6-7M	7.5	7.6	40	40	1.9
	6	2.37	-	-	-	2.6-7M	7.5	7.5			
Greenville	10	2.32	2.41	3.7	1.50						
Ave.	11	2.32	2.41	3.7	1.50	2.0-7M	6.5	6.3	33	23	0.7
	12	2.32	2.41	3.7	1.75						
	13	2.24	2.47	9.3	0.50						
	14	2.34	2.47	5.3	1.63	0	5.0	4.5	33	34	1.8
	15	2.30	2.47	6.9	0.50						

TABLE 8

MANVILLE, N. J., CORE ANALYSES, CTL REPORT 11297

Core No.	Specific Gravity at 77 F	Theoretical Maximum Specific Gravity	Air Voids (%)	Surface Course Thick. (in.)	Designed Composition		Recovered Bitumen (%)	Recovered Asphalt		
					Asbestos	Asphalt		Pen. at 77 F, 100/5	Duct. at 77 F, 5/60 cm	Ash (%)
78	2.40	2.45	2.0	1.25						
79	2.41	2.45	1.6	1.50						
81	2.37	2.45	3.3	1.50	3.0-7M	9.5	9.1	55	126	1.1
84	2.42	2.45	1.2	1.25						
87	2.38	2.45	2.9	1.25						
80	2.40	2.48	3.2	1.00						
82	2.42	2.48	2.4	1.25						
83	2.45	2.48	1.2	1.38	2.0-7M	8.0	8.7	41	97	1.5
85	2.46	2.48	0.8	1.50						
86	2.45	2.48	1.2	1.50						
88	2.31	2.59	10.8	0.88						
89	2.42	2.59	6.6	1.00						
90	2.41	2.59	6.9	1.50	0	6.0	5.2	22	19	1.2
91	2.43	2.59	6.2	0.75						
92	2.34	2.59	9.6	0.75						

TABLE 9

ST. LOUIS, MO., CORE ANALYSES, CTL REPORTS 07347 AND 08191-4

Location	Core No.	Specific Gravity at 77 F	Theoretical Maximum Specific Gravity	Air Voids (%)	Surface Course Thick. (in.)	Designed Composition		Recovered Bitumen (%)	Recovered Asphalt		
						Asbestos	Asphalt		Pen. at 77 F, 100/5	Duct. at 77 F, 5/60 cm	Ash (%)
Manchester Ave. (Tower Grove)	1	2.37	2.45	3.2	-	1.2-7M 0.6-4T	9.3	8.3		110+	2.2
	2	2.37	2.43	2.5	-	1.2-7M 0.6-4T	9.3	8.9	68	110+	1.9
	3	2.40	2.46	2.4	-	1.2-7M	9.3	8.6	60	110+	1.9
	4	2.42	2.56	5.5	-	0	6.1	5.8	54	110+	1.9
	5	2.39	2.46	2.8	-	1.4-7M 0.6-4T	9.3	8.2	60	110+	1.5
	15	2.54	2.61	2.7	-	0	6.1	5.9	40	150+	2.2
Strodtman Place	6	2.29	2.41	5.0	-	3.0-7M	11.0	9.4	48	110+	1.1
	7	2.31	2.43	4.9	-	3.0-7M	11.0	9.8	51	110+	1.4
	8	2.30	2.43	5.3	-	3.0-7M	11.0	9.3	55	110+	1.6
	9	2.30	2.41	4.6	-	3.0-7M	11.0	9.8	52	110+	2.3
	10	2.37	2.47	4.1	-	2.0-7M	9.5	8.2	48	110+	2.1
	11	2.40	2.58	7.0	-	0	6.14	5.9	25	14	2.4
	12	2.44	2.58	5.4	-	0	6.14	6.0	31	101	2.0
	14	2.41	2.57	6.2	-	0	6.14	6.1	29	64	0.9

A modification of the California water permeability test was used for measuring surface tightness of pavements in St. Louis and Manville three years after placement. The modification, as described in Appendix F, enables the solution to be forced into a 6-in. diameter area of pavement under pressures up to approximately 24 in. of water. The area of penetration, solution, time, and type of measurement are approximately the same as those used in the California permeability test.

TEST RESULTS

Visual Performance

Considering performance of the surface courses apart from structural characteristics of the pavement section, appearance is the simplest and most widely used method of evaluation. Specifically, it indicates the following:

1. Surface Toughness. — Resistance to differential wear or disintegration of the fine aggregate phase and "popping out" of coarse aggregate.

2. Stability. — Flushing or plastic displacement (rutting or shoving, etc.).

The pavements discussed in this report have been in service for 2½ to 3 years which is normally considered sufficient time to evaluate stability. In this period of time there are very noticeable differences in the surface toughness of the asbestos-modified pavements compared to the standard, although the standard pavements have satisfactory surface characteristics in most cases.

Surface Texture. — In three of the four cities, close-up photographs of the pavement surfaces have been obtained with the cooperation of sponsoring agencies (Figs. 7 through 12). At all locations, the standard pavements already show varying amounts of incipient raveling in the form of disintegration or wearing away of the fine aggregate phase, leaving the coarse aggregate projecting out above the fine aggregate phase and exposing coarse aggregate which was originally situated beneath the wearing surface. This is most pronounced in those pavements with medium traffic, where traffic densification has been least (Strodtman Avenue in St. Louis and North 13th Street, Manville) (Figs. 9, 10 and 12). The asbestos mixes with high asphalt contents show no visible evidence of surface disintegration but the Dallas pavements with the lowest asphalt content do show slight wear.

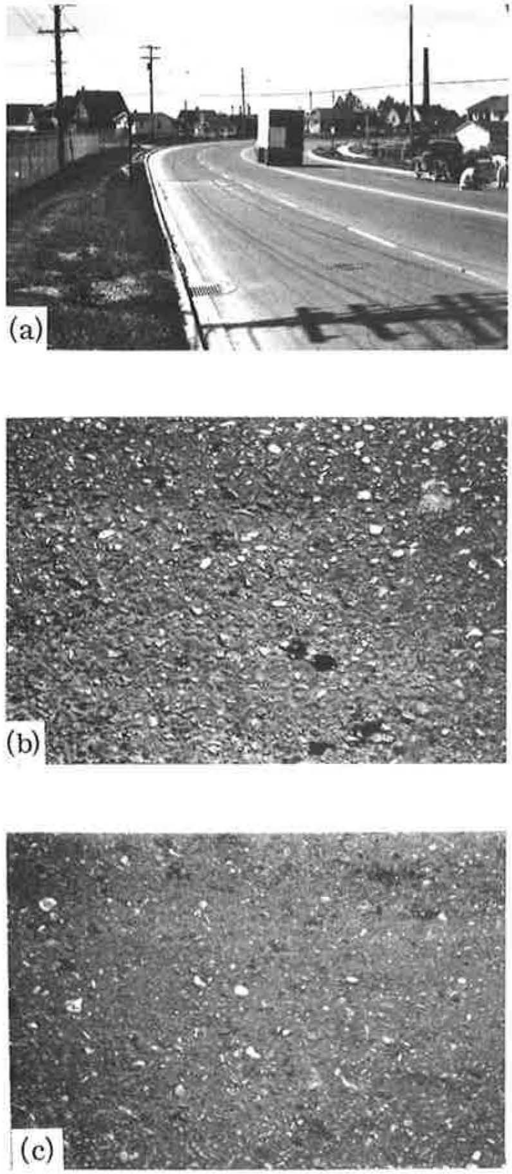

Figure 7. Alyth Freeway, Calgary, Alberta, July 1962: (a) looking south (map in Fig. 2); (b) standard pavement 5.5 percent asphalt content; (c) modification of standard mix by addition of 3 percent 7M06 asbestos and 7 to 9 percent asphalt content.

In the standard Calgary pavement with 100-120 penetration asphalt without asbestos, visible abrasion of the fine aggregate phase and exposure of the stone under heavy traffic are occurring after two years (Fig. 7b) even though penetration and ductility of the recovered asphalt are more than adequate by standard criteria. The same is true of the standard pavement (with 60-70 penetration asphalt) in St. Louis on Manchester Avenue (Fig. 11a). This differential abrasion is perhaps not severe enough to be called raveling. Nevertheless, the short asbestos fibers with the high asphalt contents appear to have prevented this abrasion of the fine aggregate phase to date in both the St. Louis and Calgary pavements (Figs. 7c and 11b). In contrast, on Strodtman Avenue the raveling evident in the standard pavement (Fig. 12b) can be explained in terms of hardness of the asphalt (Fig. 16d).

Pavement Cracks. — Figures 13 and 14 show the asbestos pavements in Manville and St. Louis, respectively; the first being a joint cracking, the second apparently a

Figure 8. Ross Avenue, Dallas, Texas, August 1962: (top) intersection with Bennett Avenue (map in Fig. 3) with standard mix 5.0 percent asphalt content in foreground and asbestos-modified pavement with 7.5 percent asphalt content at intersection; (bottom) asbestos pavement with 7.5 percent asphalt content at bus stop, opposite corner.

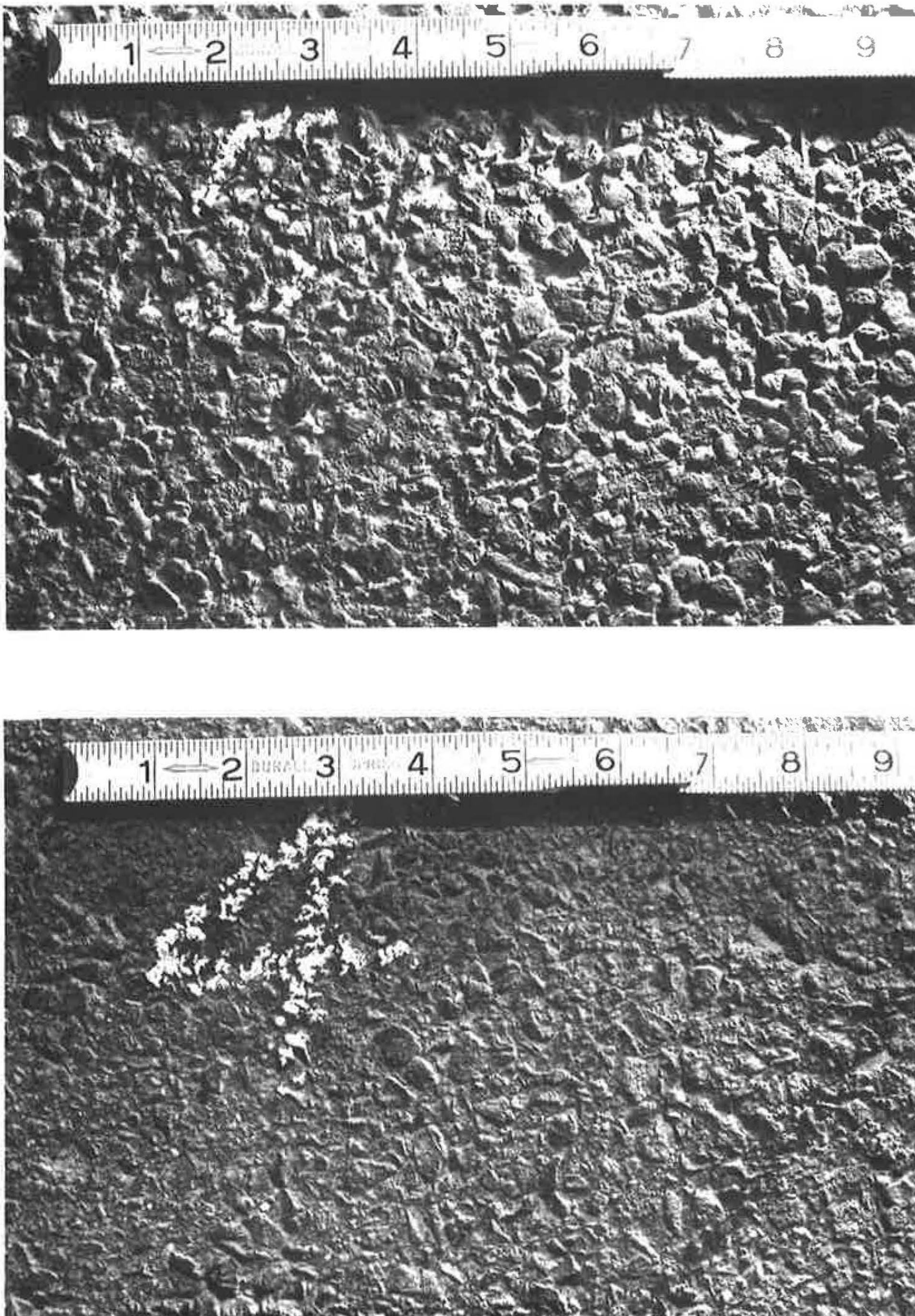

Figure 9. Variations in surface disintegration, North 13th Street, Manville, N. J., August 1962, standard pavement mix.

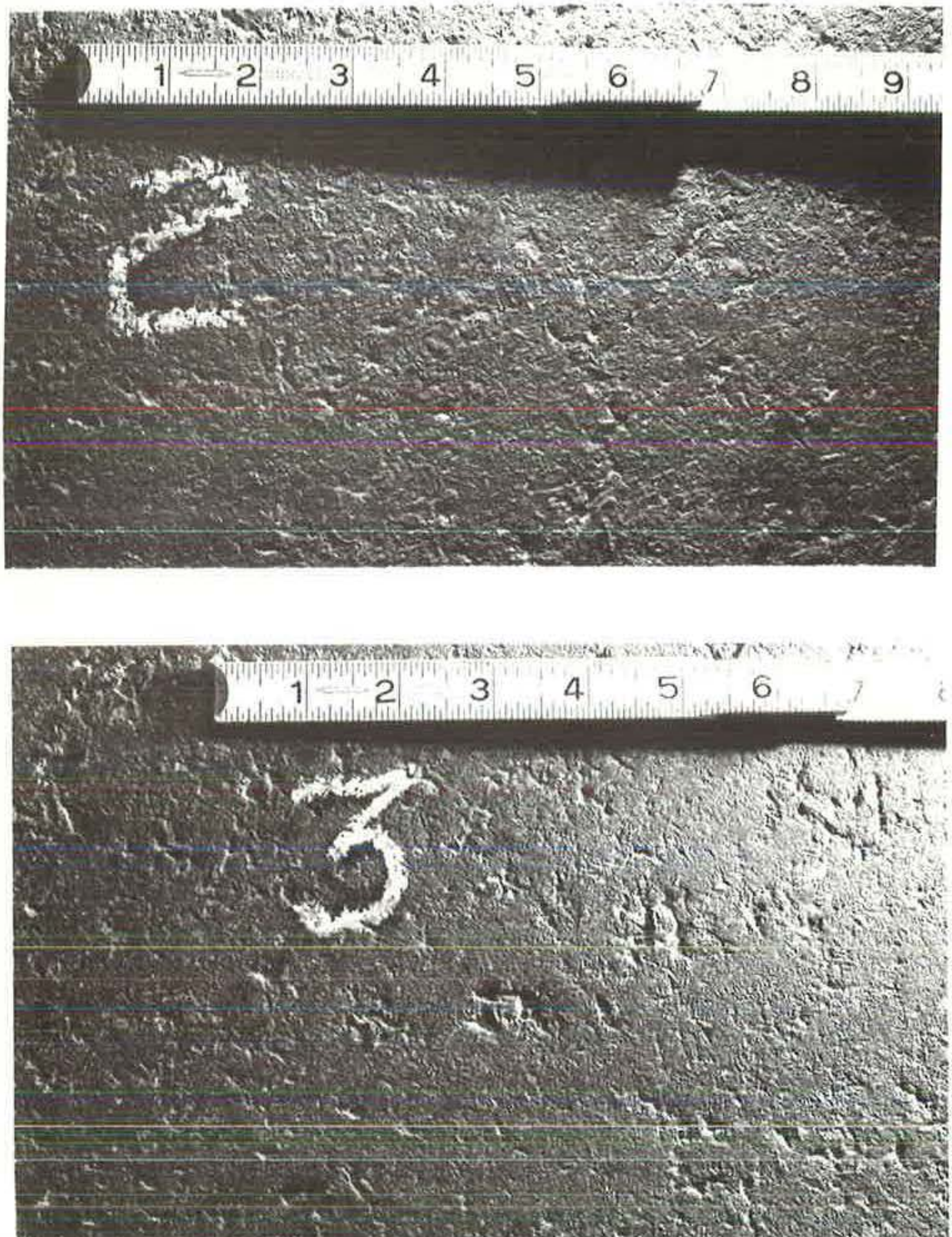

Figure 10. Variations in surface texture, North 13th Street, Manville, N. J., asbestos mix with 8 to 9.5 percent asphalt content.

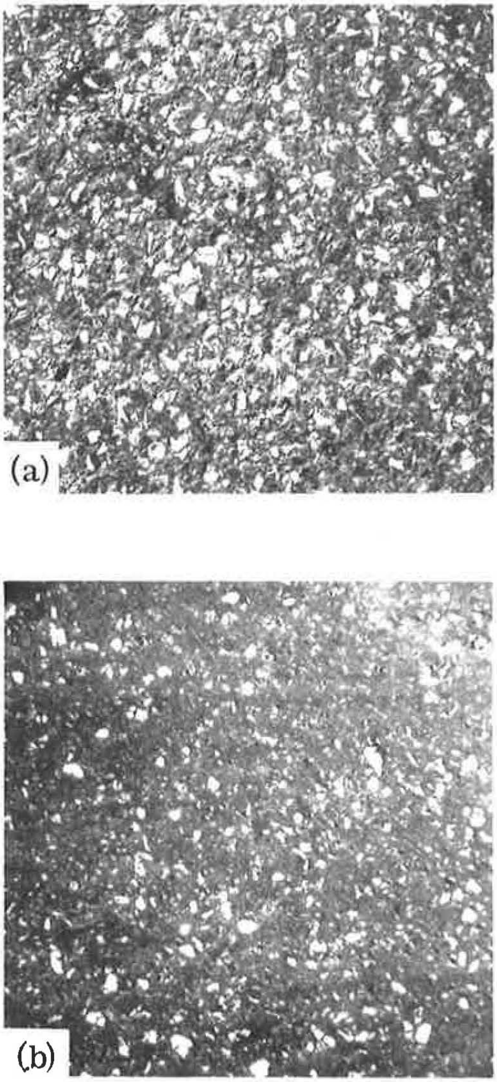

Figure 11. Manchester Avenue (near Tower Grove), St. Louis, Mo., September 1962: (a) standard pavement in traffic lane; (b) asbestos mix in traffic lane approximately 20 ft from (a).

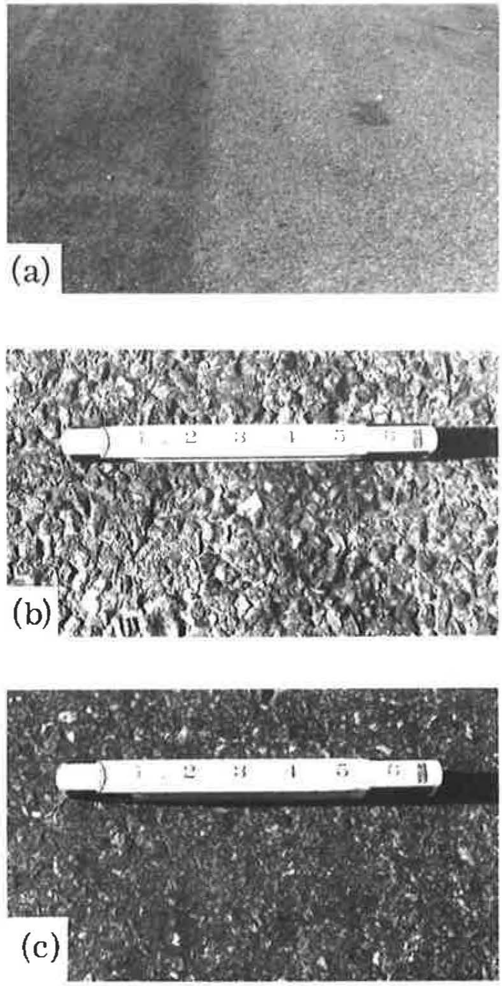

Figure 12. Strodtman Place, St. Louis, Mo., September 1962 (map in Fig. 5): (a) longitudinal boundary, asbestos on left, standard pavement on right; (b) standard pavement in traffic lane, south of asbestos section; (c) asbestos pavement in traffic lane.

reflected joint crack at the end of the new pavement. At both locations, the cracking in the standard pavement stops when it reaches the asbestos sections. Although it is not claimed that the use of asbestos will eliminate cracking, the high asphalt contents, 8 to 9.5 percent in these pavements, and the resulting high extensibility may account for this phenomenon in these cases.

It is apparent from the Dallas pavements (Fig. 15) that reflection cracking has occurred in both the standard pavement and that containing short chrysotile fibers. The cracking is linear and transverse, implying joint crackings reflected from an original portland cement pavement. The same type of reflection cracking has occurred in more recent test pavements at other locations with asphalt-concrete containing approximately 7 percent asphalt overlaid on portland cement pavements. The lower asphalt content

Figure 13. Joint crack in standard pavement stopping at south end of asbestos pavement, North 13th Street, Manville, N. J., July 1962.

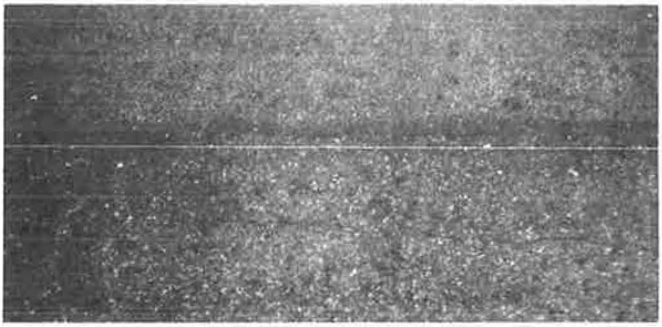

Figure 14. Reflection crack in standard mix, stopping short at longitudinal boundary of asbestos section (dark area left of center), Strodtman Place at Grand Avenue, St. Louis, Mo., September 1962.

itself compared with the Manville and St. Louis pavements may possibly account for the difference.

Stability. — With one exception, the test pavements in the four cities show no instability at present under heavy traffic, despite the fact that the asphalt content was increased 50 percent or more in at least one test pavement in each city (Table 4). The possible reasons for this are discussed in a separate section.

The exception is on Manchester Avenue in St. Louis where rutting up to 1 in. in depth is evident at Tower Grove, directly at the bus stop, and shoving occurred along a 6-ft length of curb at the Kingshighway bus stop. With the low fiber content, present knowledge indicates that the fiber-asphalt ratio of 0.19 is too low for very heavy traffic. Referring to the American Oil Company traffic simulator results (2), a minimum fiber-asphalt ratio of approximately 0.30 should be maintained (for thick overlays) to prevent the possibility of instability under very heavy traffic. It is surprising that plastic displacement is not even more severe and that no flushing or bleeding of asphalt is taking place under present traffic at these locations.

Physical Properties of the Pavements

Core Analyses — Asphalt Properties. — During the first extraction tests three years ago, the Chicago Testing Laboratory discovered that in using the standard centrifuge extraction procedure, the asbestos mixes required as much as 5 or 6 cycles to remove all asphalt, as compared with 3 cycles normally required for standard pavement specimens. Apart from core analyses, a number of test series have been carried out by the Chicago Testing Laboratory on pavement specimens made in the laboratory with specific asphalt contents, from which completeness of asphalt and properties of asphalt could be checked. Results of two of these series, Shattuck oxidation tests and aging tests, which include a wide variety of asphalt sources, are given in Appendixes A and B. Their tests of this type suggest that for the field core analyses performed by the Chicago Testing Laboratory:

1. The recovery of asphalt from the asbestos and standard mixes is equivalent and essentially complete, within the limits of accuracy of the procedure.
2. The effect of very small amounts of retained asphalt (adsorbed on the filler and fiber surfaces or even within the pores of the rock aggregate) on physical properties of recovered asphalt, if any, is the same in both the standard and asbestos mixes.

The authors conclude that the asbestos fibers had no significant differential effect on consistency of asphalt recovered from the test pavements described herein.

Figure 15. Reflection cracking, Ross Avenue, Dallas, Texas, August 1962.

Because the life of all asphalt pavement surface courses is limited by weathering of the binder and progressive loss of cohesive strength, it follows that some indications of ultimate performance may be obtained by core analysis before the extensive deterioration of the surface has occurred. Detailed core analysis data taken from the Chicago Testing Laboratory reports are given in Tables 6 through 9. Table 10 summarizes asphalt contents from core analyses. Comparisons of penetration and ductility of recovered asphalt are shown in Figures 16 and 17.

The average penetration of asphalt recovered from the asbestos pavements is 80 percent higher than that from the standard pavements. In two of the asbestos pavements the recovered asphalt showed more than 90 percent of original penetration, whereas in three of the four standard mixes, asphalt penetration of recovered asphalt was critically low by past performance standards. The ductility of asphalt from three of the four standard mixes was markedly reduced below original values compared to the corresponding asbestos mixes. In only one of the eight asbestos mixes was hardening of the asphalt equivalent to that of the standard pavements. This was the Greenville Avenue pavement in Dallas, which contained the lowest asphalt content (6.5 percent) of the eight asbestos mixes being studied.

There are at least two basic causes for reduction in penetration and ductility of asphalt recovered from asphalt pavements:

1. Hardening during mixing (volatilization and high-temperature oxidation).
2. Hardening due to weathering processes (oxidation-polymerization).

The large increases in asphalt content in the asbestos pavements may have affected both hardening processes favorably through the thicker asphalt coatings on the aggregate particles and by facilitating low initial void contents. The total effect is perhaps best illustrated by the core analyses of the traffic simulator test pavements (2) (Fig. 18) which included the widest range of asphalt for both standard and asbestos mixes (Illinois I-11). From the relatively low penetration of recovered asphalt, it appears that the radiation lamps used to heat the pavements successfully induced accelerated weathering.

The asbestos mixes at 5 and 6.5 percent asphalt content should not be compared with the standard pavements with respect to penetration of recovered asphalt because the former

TABLE 10

ASPHALT CONTENTS

Location	Mix No.	Asbestos Content (% total weight)	Asphalt Contents, % Total Weight of Mix: Plant Mix Analysis or Production Record: Asphalt Pen.	Std Mix	Asbestos Mix	Increase Above Standard %	Recent Core Analyses: Std. Mix	Asbestos Mix: Set 1	Asbestos Mix: Set 2	Increase Above Std.%: Set 1	Increase Above Std.%: Set 2
Calgary	1	3.0	100-120	5.5	9.2	67	5.3	8.0	7.2	51	36
	2	3.0	100-120		7.0	27					
								1962	1961	1962	1961
Dallas	1	1.7	85-100	5.0	6.5	30	5.0	6.5	6.9	30	38
	2	2.6	85-100		7.5	50		6.9	7.5	38	50
Manville	1	2.0	85-100	5.9	8.0	36	5.2	8.7		67	
	2	3.0	85-100		9.0-9.5	53-61		9.1		75	
St. Louis	1	1.8	60-70	6.2	9.3	50	5.7	8.5		49	
	2	2.0	60-70		8.4	35	5.9	8.2		39	
	3	3.0	60-70		9.8	58	5.9	9.6		63	

sustained at least one million more wheel passes at temperatures 20° higher than the standard pavements. It appears that for both the standard and asbestos pavements, as asphalt content is increased, the total hardening of the asphalt (during mixing and testing) is sharply reduced. This is, of course, a most desirable effect.

Figure 19 shows a comparable effect in the Calgary pavements (Table 6), both of which contain 3 percent asbestos and equivalent void contents. The Manville pavements, which have equivalent void contents, show the same general effect of increased asphalt content on asphalt hardening (Fig. 16c). At other locations, the effect of the difference in void content on asphalt hardening is apparent.

The relationship between asphalt content and penetration of recovered asphalt is not linear. The core analyses suggest that where asphalt content was increased significantly above 7 percent (total weight of mix), recovered asphalt penetration was high. At 7 percent asphalt content or below, the differences in penetration are usually not evident.

With regard to weathering, the Maryland roads study (7) suggests that the rate of oxidation after placement for a given type of asphalt depends on exposure of the pavement to air in the voids. This effect of void contents on hardening shows up in some of the core analyses; for instance, the St. Louis pavements. The Manchester Avenue pavements, both standard and asbestos, show a higher penetration of recovered asphalt than the corresponding Strodtman Avenue pavements despite the much higher asphalt content in the latter asbestos pavements (Fig. 16). The probable cause is the much higher void contents in the Strodtman Place pavement than in the Manchester Avenue pavements where the traffic was not heavy enough to reduce the void content to a low level.

Void Contents and Resistance to Compaction. — At standard asphalt contents or at asphalt contents reduced below standard, it was first shown by the Asphalt Institute (8) that short asbestos fibers produce an outstanding resistance to compaction. This was confirmed by full-scale traffic simulator tests by the American Oil Company (2). No measurable reduction was shown in air void content in the pavement containing 2½ percent asbestos at 5.0 percent asphalt content with either 85-100 penetration or

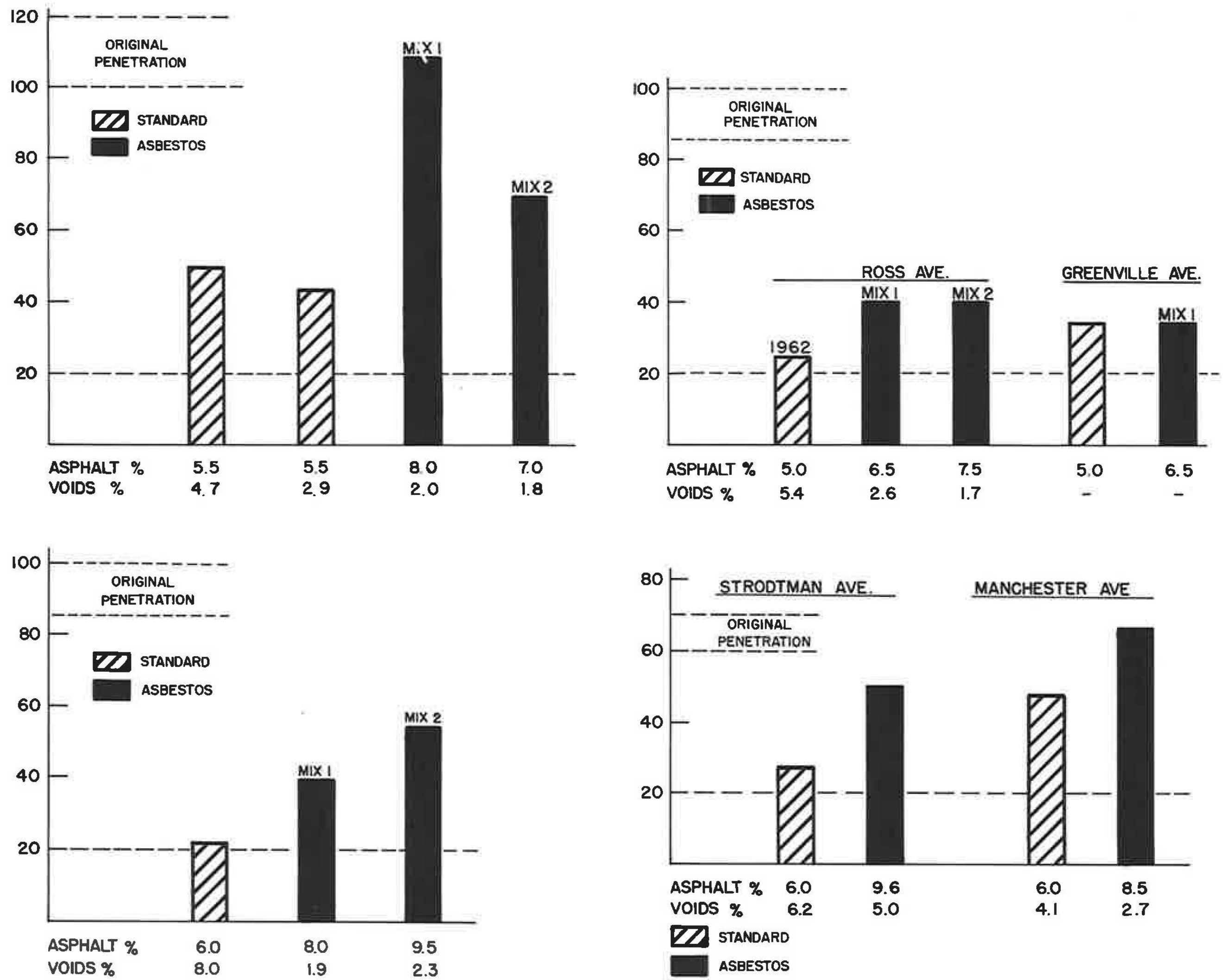

Figure 16. Penetration of recovered asphalt: (a) Calgary, Alberta; (b) Dallas, Texas; (c) Manville, N.J.; (d) St. Louis, Mo.

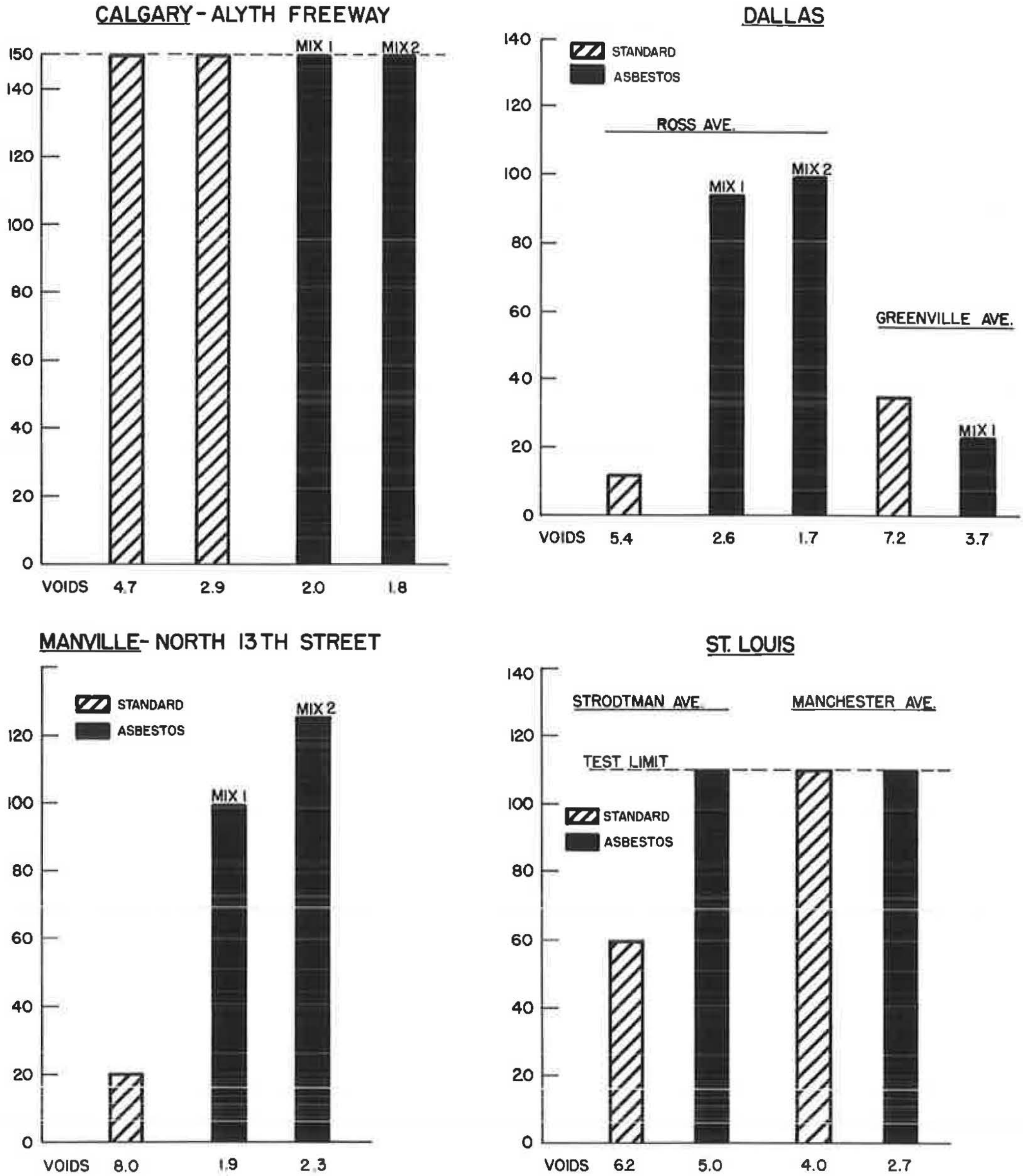

Figure 17. Ductility of recovered asphalt.

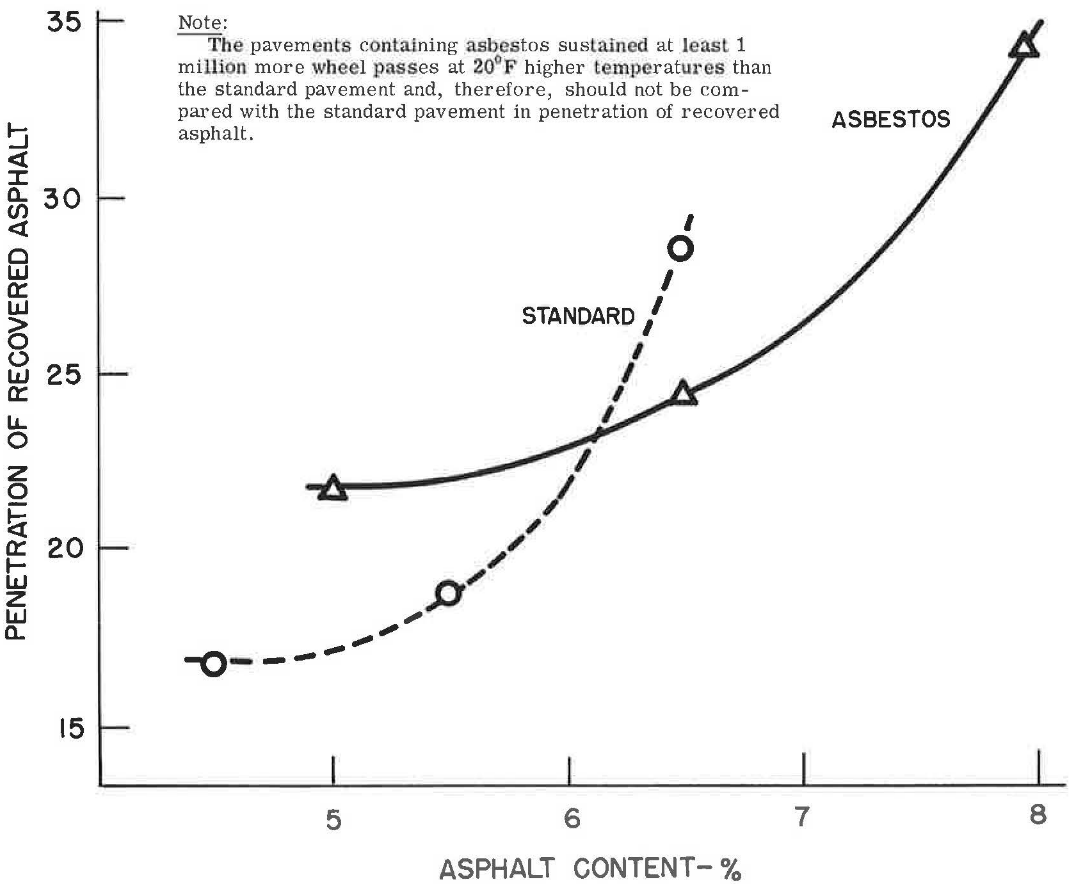

Figure 18. Effect of asphalt content on hardening of asphalt, traffic simulator tests, original penetration 60-70. Pavements containing asbestos sustained at least 1 million more wheel passes at 20 F higher temperatures than standard pavement and therefore are incomparable with it in penetration of recovered asphalt.

60-70 penetration after three million wheel passes duplicating heavy truck traffic. At high asphalt contents, both types of tests showed greatly reduced resistance to compaction.

Because low void contents are beneficial from the standpoint of pavement durability, perhaps the pertinent point is whether the test pavements have been overcompacted under traffic and become unstable. Core data obtained in 1960 and 1962 from pavements with high asphalt content and sufficient asbestos show that where initial void contents were very low (as in Calgary and Manville) no measurable reduction in voids has occurred under traffic and no instability is evident to date despite an increase in asphalt content of 50 percent above standard optimum.

The Calgary core data are given in Table 6 and Appendix C. The increase in void content since placement, implied by the data, is probably due to differences in sample size, area cored, and methods of laboratory analysis. The cores removed from between the wheel paths (center) showed slightly but consistently lower voids than those taken in the right and left wheel paths. The Manville core data are given in Table 11.

Reference to individual core results illustrates another difference. In addition to consistently lower void contents, the range or variability in void contents for the

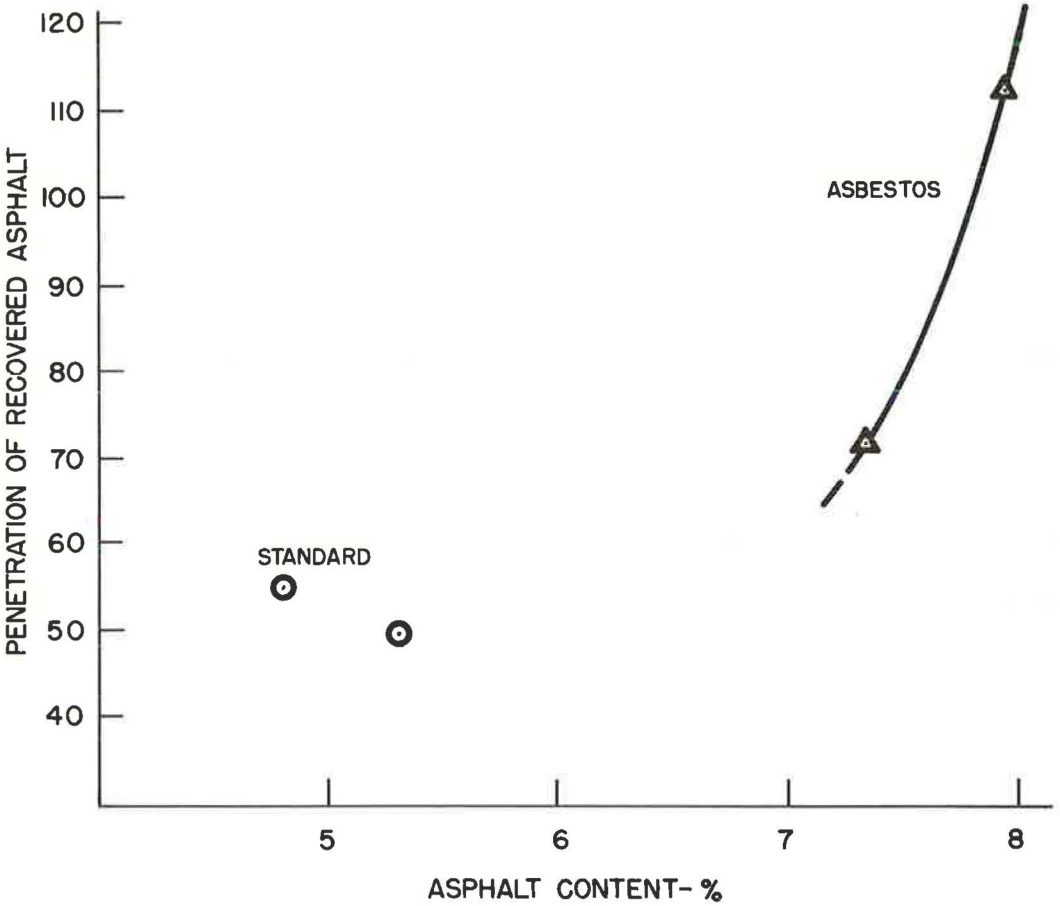

Figure 19. Effect of asphalt content on hardening of asphalt, Calgary, Alberta, original penetration 100-20.

asbestos mixes at each location was consistently one-half (or less) than that of the adjacent standard mix. This uniformity of compaction is, of course, very desirable.

Water Permeability. — Figure 20 shows the general relationship between pavement permeability and air voids as illustrated by data from the Manville pavement. Above a critical air void content (in this case, approximately 6 percent), permeability increased rapidly. Below this void content, permeability was negligible. The two-phase relationship between void content and permeability may be related to interconnection of voids. Further, the test was capable of measuring the slight differences between the pavements with 2 and 3 percent asbestos content.

Average water permeability data from the St. Louis pavements are given in Table 12. Air permeability tests performed at the same locations show a good correlation with water permeability. Both tests indicate that below a critical void content, somewhere between 5 and 7 percent, permeabilities are negligible.

These data suggest that although the asbestos pavements were very tight, small but measurable permeabilities were obtained, with the exception of the Manchester Avenue pavement in St. Louis, which is judged to be too low in fiber content for the existing traffic loads.

The main purpose of these tests is to determine whether a relationship exists between the durability of wearing courses and permeability. This will require periodic retesting for permeability and simultaneous performance evaluations in the future.

Skid Resistance Properties. – The first asbestos test pavement with high asphalt content was placed in Manville on North 13th Street in September 1959. Because considerable flushing occurred under the roller in Mix 2, the skid resistance properties of the pavement were checked to measure the effect, if any, of asphalt contents increased from 5.9 to 9.5 percent total weight.

A simple skid resistance test was first used, in which a passenger car was towed at slow speeds over a wet pavement and the force required to pull the car with wheels locked was recorded to determine the coefficient of dynamic friction. These tests showed no significant difference in skid resistance between standard pavement mixes and the two asbestos pavements.

TABLE 11

MANVILLE, NORTH 13TH STREET, CORE DATA

Property	1960 Coring	1962 Coring
Chicago Testing Laboratory report	No. 08411	No. 11297
Number of cores	3	10
Air void content (% by volume):		
Asbestos mix I	1.2	1.9
Asbestos mix II	2.3	2.3

In 1960, one year after placement, a second series of tests was performed at 30 mph using the standard stopping distance tests with a Wagner Stopmeter and fifth wheel

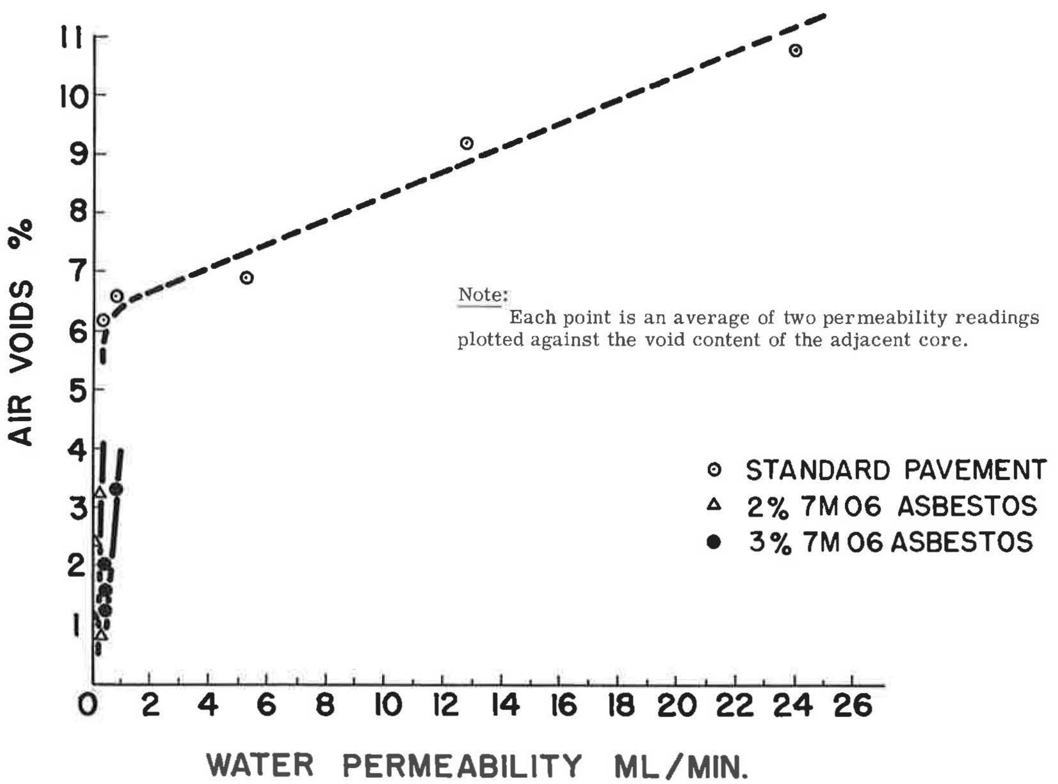

Figure 20. Pavement permeability vs air voids, North 13th Street, Manville, N. J. Each point is average of two permeability readings plotted against void content of adjacent core.

TABLE 12

WATER PERMEABILITY OF ST. LOUIS PAVEMENTS

Location	Void Content (% by volume)		Permeability[a] (ml/min)	
	Standard	Asbestos	Standard	Asbestos
Strodtman Place	6.2	5.0	0.3 to 4.7	0.1 to 0.2
Manchester Avenue	4.1	2.7	0 to 4.0	0

[a]Two test locations were used for each pavement mix.

TABLE 13

STOPPING DISTANCE TEST RESULTS, 1960
NORTH 13TH STREET, MANVILLE, N. J.

Property	Test No.	Stopping Distance at 30 Mph (ft)	Coefficient of Friction	Average Deceleration (ft/sec)	Condition
Standard mix	1	61	0.49	15.9	Wet
New Jersey standard mix	2	66	0.455	14.6	Rewet
Asbestos modification:					
Mix 1	1	68	0.44	14.2	Wet
	2	67.5	0.445	14.3	Rewet
Mix 2	1	66	0.455	14.6	Wet
	2	68	0.44	14.1	Rewet

attached to a passenger car. The results in Table 13 show again that there was a slight but insignificant difference between the stopping distances of the standard New Jersey SM pavement and the asbestos-modified mixes with high asphalt contents. These tests were performed by Johns-Manville friction materials engineers who concluded from the tests that "friction coefficients showed no significant or consistent relationship to asphalt contents of the test roads." Detailed results, including calculation of coefficient of friction, are given in the table.

The design standard for safe pavements adopted by the American Association of State Highway Officials provides for a maximum stopping distance of 113 ft from 40 mph on a wet pavement, which corresponds to 64 ft at 30 mph. The average stopping distances obtained on the Manville, North 13th Street, pavements are close to this standard.

It has often been shown that polishing of various types of aggregates by traffic or inherent smoothness of gravel is the main cause of progressive deterioration of skid resistance in many standard asphalt pavements. A recent report (11) on pavement slipperiness in Virginia states,

> Although the type of pavement plays an important role in slipperiness during the early life of a pavement, once the cement mortar or bituminous binder has worn from the surface, it is the (aggregate) materials that make up a pavement which determines whether a highway will polish and become slippery.

Because the fine asbestos fibers become an integral part of the binder, the fibers themselves would not be expected to affect skid resistance directly. However, as shown in the foregoing photographs of the pavement surfaces, the increased surface toughness of the asbestos-asphalt pavement with high asphalt contents should help inhibit deterioration of skid resistance by preventing differential abrasion of the fine aggregate and exposure of the susceptible coarse aggregate to tire traffic. Future skid tests will be performed to measure the relative effect of abrasion and aggregate polishing on these test pavements with high asphalt content and adjacent standard pavements.

Pavement Life Predictions

The durability study of Maryland pavement surfaces reported by Goode and Owings (Fig. 1) shows that the initial 4-year hardening rate was proportional to the initial air voids. Their data also suggest that hardening rate decreases as void content decreases, implying that an estimate of relative time required for the pavement to reach a critically low penetration or loss of ductility might be predictable.

In the present report, the average difference in present void content is shown in Figure 21.

These differences in air voids, an average of approximately 50 percent lower voids in the asbestos pavements, undoubtedly account for much of the present differences in penetration and ductility of recovered asphalt shown in Figures 16 and 17.

Taking the core analysis data from the Calgary pavements, which remain well above the critical penetration value of 20, described in past pavement studies, it is possible to estimate the relative life expected in the two types of surface mixes or at least the relative time required for the pavements to reach a critically low penetration

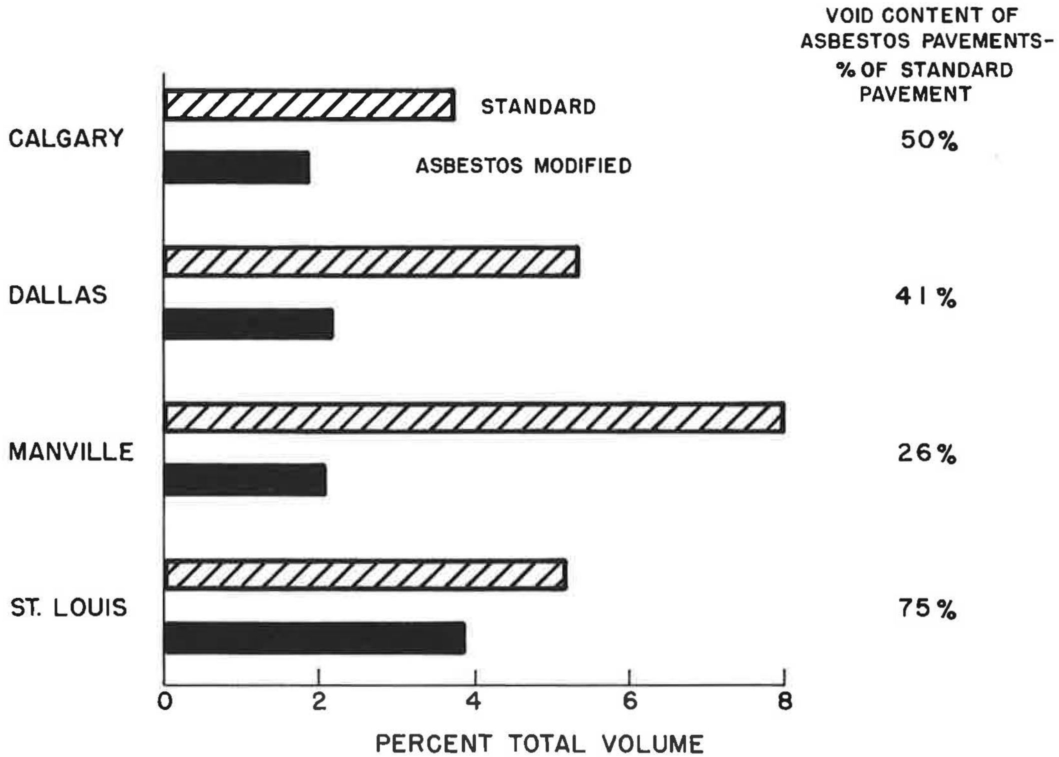

Figure 21. Average void content.

of 20. First, ignoring the present difference in void contents between the standard and asbestos pavements, based on present asphalt penetrations, 30 and 71, respectively (Fig. 16), it should take more than twice as long for the asbestos pavement to reach the critical 20 penetration, assuming the same rate of asphalt penetration loss. However, the standard pavement at present shows twice the void content of asbestos sections and it might therefore be concluded that the rate of penetration loss in the standard pavement would be twice as great as the average rate of loss in the asbestos sections. Allowing for greater future compaction in the standard mix than in the asbestos pavement, it still might reasonably be concluded that the predicted life of the asbestos pavement will be from 100 to 200 percent longer than the standard surface mix. By the same process, it can be shown that the predicted life of the asbestos surface pavement on Manchester Avenue in St. Louis could be twice that of the standard pavement. It should be recalled that at both of the preceding locations visible abrasion has already taken place in the standard pavements, although asphalt penetration and ductility were not critically low.

The service life of most asphalt pavements is determined by (a) structural performance of the pavement section, and (b) the durability of the surface course; that is, the resistance of the surface course to the combined effects of weathering and traffic. Undoubtedly, structural performance, as regards cracking, is to some extent dependent on the durability of the surface course. In some places the structural design is deliberately limited to the expected life of the surface course because resurfacing will upgrade the pavement structurally (12). However, for many asphalt pavements it appears that "structural life" far exceeds the "durability life" of the surface course.

In these pavements there is good reason to increase service life of the surface course in the original pavement so that it equals anticipated structural life. Use of asbestos with high asphalt in the surface course of new pavement should help make this performance equality possible.

The increased cost of any modified product must be compared to the improved performance expected. It can be shown that the increase in cost of adding short chrysotile fiber to the top 1 to $1\frac{1}{2}$ in. of surface course is approximately \$0.10 to \$0.15 per square yard, including increased asphalt content.

Mix Design Tests

Standard Design. — In Figure 22 and Table 14, recent mix design test results are

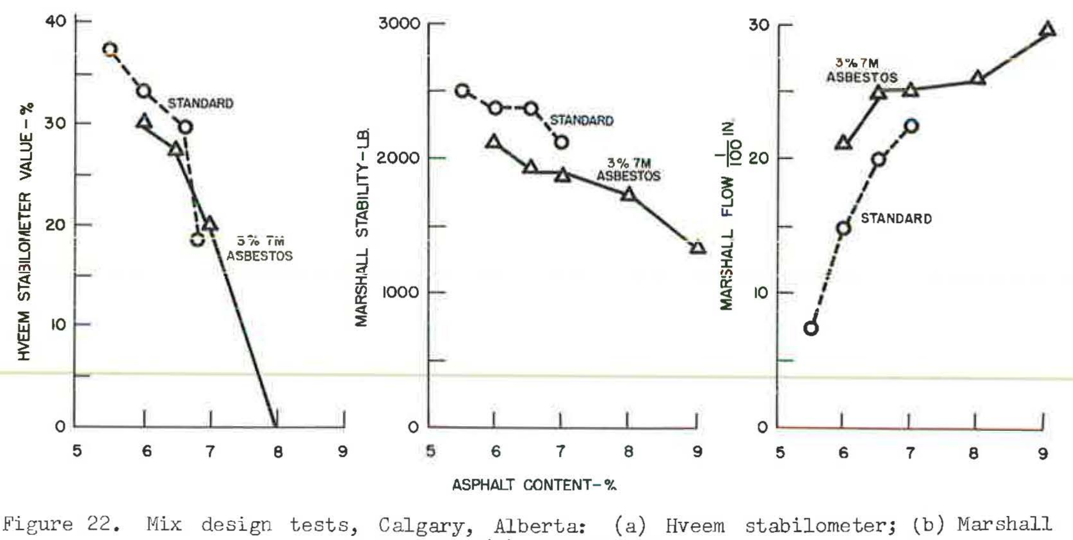

Figure 22. Mix design tests, Calgary, Alberta: (a) Hveem stabilometer; (b) Marshall stability; (c) Marshall flow.

TABLE 14

CALGARY LABORATORY TEST DATA

Sample No.	Asphalt Content (%)	Fiber Content (%)	Density (g/cc)	Void Content (%)	Marshall Stability (lb)	Marshall Flow (0.01 in.)	Hveem Stabilometer Value (%)
10	5.5	0	2.38	2.21	2,550	7	38
11	6.0	0	2.39	0.87	2,420	15	34
12	6.5	0	2.40	-0.41	2,400	20	30
13	7.0	0	2.39	-0.48	2,100	23	19
14	6.0	3	2.30	4.75	2,100	21	30
15	6.5	3	2.32	2.90	1,960	25	28
16	7.0	3	2.34	1.26	1,910	25	20
17	8.0	3	2.33	0.19	1,720	26	0
18	9.0	3	2.30	-0.31	1,380	30	0

shown for aggregates and asphalt received from Calgary. The mix duplicates that used in the test pavement placed in 1960. The specimens were compacted by a gyratory compactor following Asphalt Institute procedure rather than standard Marshall and Hveem methods and, therefore, the actual stability values given are not equivalent to standard test results. Nevertheless, it is apparent that the high asphalt contents permitted in asbestos-asphalt pavements above 7 percent (as demonstrated by this report) could not have been predicted from standard mix design test criteria. The stability tests performed on plant mixes and cores in Calgary and St. Louis by an independent laboratory shown in Appendixes C and D support this conclusion. The curves in Figure 22 are typical of tests on asbestos-asphalt concrete published previously by other laboratories.

It is generally agreed that the stabilometer test measures almost exclusively the aggregate interlock with little effect shown by the cohesion of the binder. Although the Marshall test data are thought to measure a combination of aggregate interlock and cohesion, the main configuration of the curve, if not the stability values, shown in Figure 22b (i.e., optimum asphalt content, etc.) also appears to be controlled by aggregate interlock.

In Figure 22a, above 7.5 percent asphalt contents, the stabilometer values of the Calgary pavement mixes with and without asbestos are negligible. Yet, the Calgary pavement with asphalt contents from 8 to 9 percent containing 3 percent asbestos has remained stable under heavy traffic. The same comparison was observed in the traffic simulator test report (2) and in other high asphalt pavements with asbestos placed for field evaluation. Apparently, performance of the asbestos mixes is not controlled by aggregate interlock as measured by the stabilometer test.

In most pavement mixes, standard mix design compaction procedures give much higher densities and lower void contents than those produced by field compaction of the identical asbestos-asphalt mixes. Correlations between laboratory and field compaction and related mix design criteria established for standard pavements apparently do not apply and should not be used in design of asbestos-asphalt pavements to determine optimum asphalt content or field compaction characteristics.

Mix Design of Asbestos-Asphalt Surface Courses. — The purpose of mix design for standard asphalt surface courses is to determine what is called "optimum asphalt content." The original empirical criteria used to obtain these optimum values imply that the objective is to find the maximum asphalt content that can be used without instability in service. Arbitrary use of asphalt content somewhat below optimum is sometimes used as a safety factor, but little if any increase in asphalt content is permitted.

The asbestos-asphalt pavements placed in each of the four cities, previously listed, include at least one with a 50 to 60 percent increase in asphalt content above standard optimum (Table 10). Apparently, for the typical surface mix gradations represented by these pavements, asphalt content may be chosen independently of standard mix design criteria. These increases would appear to be as much or more than would ever conceivably be needed. This, in effect, adds a new "dimension" to pavement design; i.e., asphalt content. Therefore, other desirable objectives for mix design can be used, most of which relate directly to increased asphalt content, including resistance to asphalt hardening, minimum void content, low temperature impact strength, flexural fatigue strength, etc.

The explanation of how the fine asbestos fibers permit the large increase in asphalt content is beyond the scope of this report. However, there is evidence for at least two basic strength mechanisms: (a) resistance to flow of asphalt by inter-fiber bonding (2) and/or fiber orientation, and (b) resilience of the fiber to asphalt mastic films (10, 13).

Field performance is perhaps the final basis for design. Beginning with asphalt content, the performance data for asbestos-asphalt pavements previously described suggest that a 35 percent increase in standard asphalt content is needed to guarantee a marked increase in pavement life (Figs. 18 and 19, and previous discussion of properties of recovered asphalt). For these typical standard surface courses with 5.5 to 6.0 percent optimum asphalt content, asphalt content should be at least 8 percent total weight of the mix when short asbestos fibers are added. Selection of asbestos content could then be based on minimum fiber-asphalt ratio necessary to maintain adequate stability.

At the 1962 Highway Research Board annual meeting, results of full-scale traffic simulator tests on asbestos-asphalt pavements were reported by the American Oil Company (2). In these tests, heavy wheel loads were applied on pavement slabs for a total of $2\frac{1}{2}$ million wheel coverages at temperatures from 90 to 135 F. The Illinois surface mix with $2\frac{1}{2}$ percent asbestos and 8 percent asphalt content was far superior in performance to the control mix containing 5 percent asphalt (below optimum asphalt), the best standard mix in performance (Figs. 23 through 26). The critical fiber to asphalt ratio for this mix was approximately 0.31.

The field performance of the asbestos-asphalt pavements placed since 1959, including those described in this report, confirms the existence of a critical fiber to asphalt ratio between 0.25 and 0.30 for heavy traffic. Based on present satisfactory performance of pavements under heavy, medium, and light traffic, safe fiber to asphalt ratios for surface courses would be 0.30, 0.25, and 0.20, respectively. For typical surfaces previously described, the corresponding fiber contents with 8 percent asphalt content would be approximately 2.5, 2.0, and 1.6 percent total weight of mix, respectively.

As shown previously, visible surface abrasion was evident in two standard pavements even though the asphalt had not hardened excessively (St. Louis and Calgary pavements). To date, the adjacent asbestos pavements with high asphalt content have resisted this abrasion through increased cohesion which in most cases appears to be proportional to the asbestos content (up to 3 percent). In the preceding mix design recommendations this factor is accounted for inasmuch as the heavy traffic pavements most susceptible to traffic abrasion require higher fiber contents to maintain stability.

The recommended asbestos and asphalt contents should be satisfactory with asphalts up to 100 penetration, with the exception of West Coast States where no asbestos-asphalt pavements with high asphalt contents have been placed from which to judge.

Recent commercial use of asbestos-asphalt pavements include successful use of lower asphalt and fiber contents than those recommended. The main objective is to improve cohesion of those pavements which in performance life are below the average standard asphalt pavement. Among these are thin overlays and pavements made from aggregates lacking in fines, etc. Also, some of these overlay pavements are much finer in aggregate gradation than the pavements described in this report and conclusions given do not necessarily apply to the finer gradations.

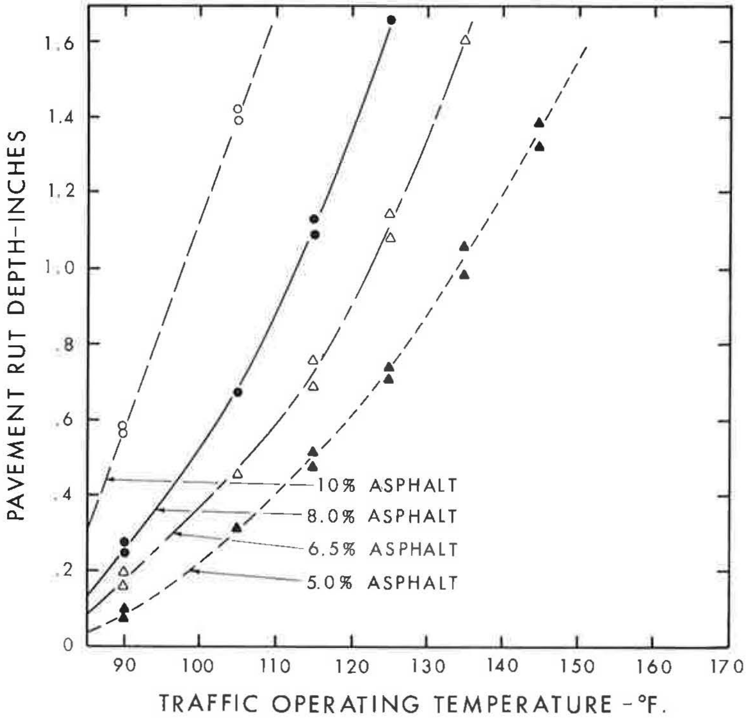

Figure 23. Performance of control pavements with 85 penetration asphalt (from 2).

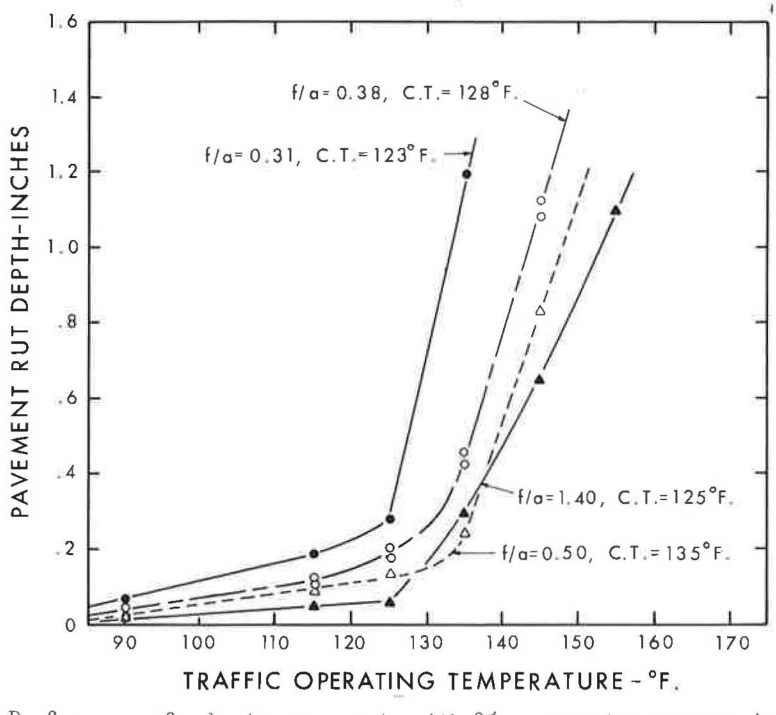

Figure 24. Performance of asbestos pavements with 85 penetration asphalt (from 2).

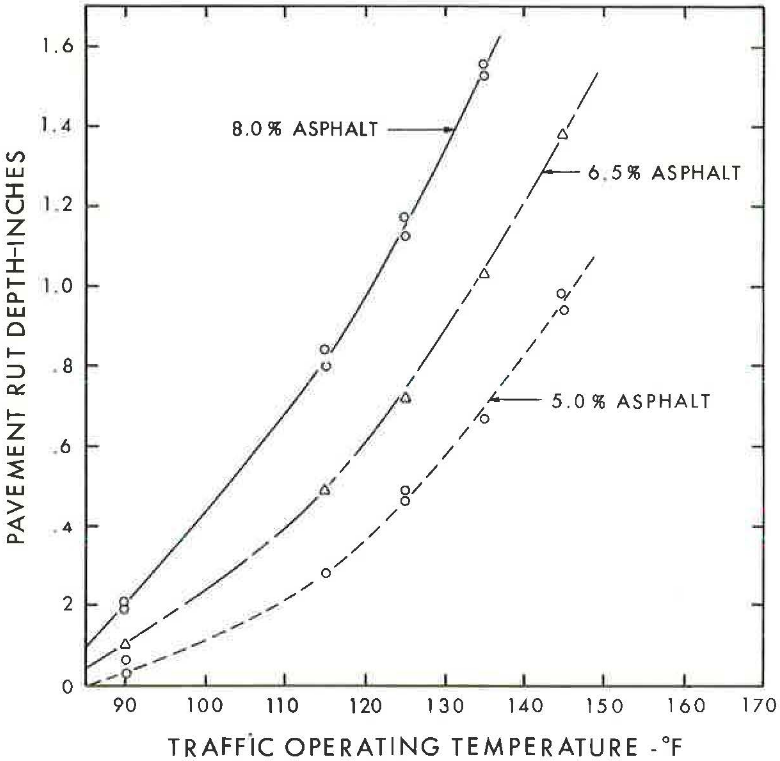

Figure 25. Performance of control pavements with 60 penetration asphalt (from 2).

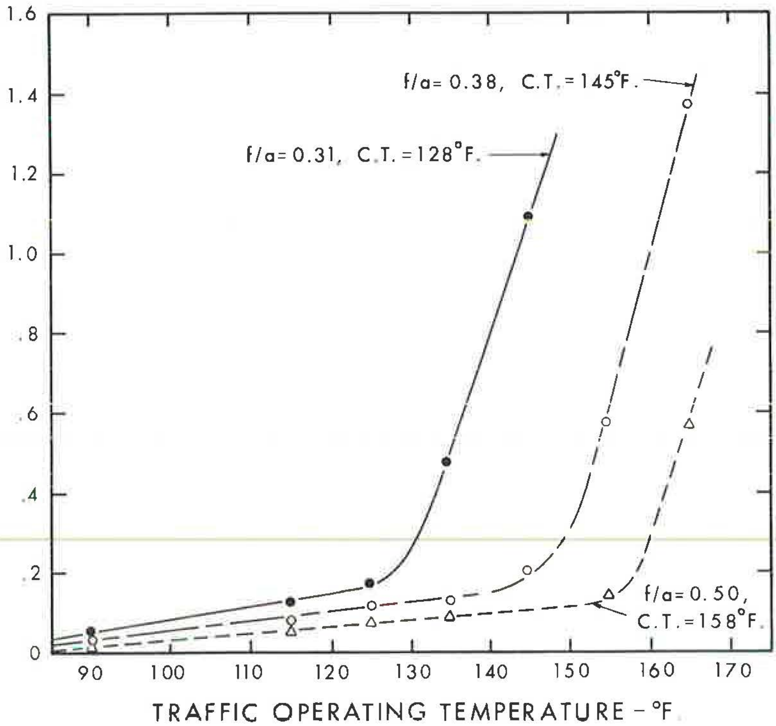

Figure 26. Performance of asbestos pavements with 60 penetration asphalt (from 2).

SUMMARY AND CONCLUSIONS

The following tentative conclusions are based on $2\frac{1}{2}$- to 3-year performance of eight asphalt pavements containing Johns-Manville 7M06 asbestos placed in four cities. All these pavements were modifications of standard dense-graded surface mixes $1\frac{1}{4}$ to $1\frac{1}{2}$ in. in thickness containing approximately 50 to 60 percent stone retained on the No. 8 mesh sieve.

1. With correct fiber to asphalt ratios, asphalt contents can be increased 50 percent or more above standard optimum without instability under heavy traffic.
2. Core analysis indicates that, where initial void contents were low, no significant reduction in air voids under traffic occurred.
3. Where standard asphalt content was increased 35 percent or more in the asbestos mixes, hardening rate of the asphalt was markedly reduced.
4. In two cities, more than 90 percent of the original asphalt penetration value still remains in the asbestos-asphalt pavements.
5. Incipient raveling of varying degree, already evident in most of the standard mixes, has not occurred in adjacent asbestos-modified pavements.
6. Hardening of the asphalt appears to be responsible for incipient raveling or surface abrasion in the standard pavements at two locations.
7. In at least two cities, surface abrasion of the standard pavements has occurred even though the asphalt penetration is not critically low. The toughness of the adjacent asbestos-asphalt surface mixes has prevented this type of surface deterioration.
8. The permitted 50 percent increase in asphalt content demonstrated by these asbestos-asphalt pavements could not have been predicted by standard mix design tests. Optimum asphalt content of asbestos-asphalt pavements is not limited by standard mix design criteria.
9. Concerning stability in service, the controlling factor in mix design appears to be fiber to asphalt ratio and not asphalt content as in standard pavement mixes.
10. Fifth wheel stopping distance tests on the Manville pavements showed no reduction in skid resistance when asphalt content was increased from 5.9 percent in the standard mix to 9.0 percent in the asbestos pavements.

Based on these performance data, general mix design recommendations, including fiber and asphalt contents, are given for the typical aggregate gradation represented by these surface courses.

ACKNOWLEDGMENTS

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The assistance of Johns-Manville's representatives in various offices was invaluable. Assisting in preparation of the report were D. A. Tamburro, W. A. Riaski, and C. F. Greene. D. D. Bates assisted in the photographic work. The cooperation of the Packings and Friction Materials Department at the Johns-Manville Research Center in performing stopping distance tests is greatly appreciated.

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Appendix A

SHATTUCK OXIDATION TESTS—EFFECT OF ASBESTOS ON THE HARDENING OF ASPHALT DURING MIXING

Laboratory. — Chicago Testing Laboratory, Report 03886, February 2, 1961.
Test Method. — Shattuck Oxidation Tests Described in AAPT Proceedings, 11: 193-194 (1940).
Materials. — Johns-Manville's 7M06 asbestos
Ottawa silica sand
Asphalt from six sources (Table 15)

TABLE 15

RESULTS OF SHATTUCK OXIDATION AND RECOVERY TESTS

Asphalt Crude Source	Original Asphalt Cement		Recovery After Regular Shattuck				Recovery After Shattuck with 2.5% Asbestos			
	Pen. at 77 F	Duct. at 77 F	Pen. at 77 F	% of Orig.	Duct. at 77 F	Ash (%)	Pen. at 77 F	% of Orig.	Duct. at 77 F	Ash (%)
Boscan	91	150+	36	39.6	63	0.01	38	41.8	55	1.7
Wyoming	93	150+	47	50.5	110+	0.02	46	49.5	110+	1.9
Mid. Cont.	88	150+	47	53.3	22	0.01	49	55.7	24	2.0
Smackover	82	150+	48	52.8	110+	0.01	47	51.7	110+	1.5
E. Texas	91	150+	39	42.8	68	0.03	38	41.8	55	1.6
Oklahoma-solvent ppt.	95	150+	57	60.0	92	0.03	59	62.2	145	1.3

Appendix B

EFFECT OF AGING ON THE PROPERTIES OF ASPHALTIC-CONCRETE MIXTURES CONTAINING ASBESTOS

Laboratory. — Chicago Testing Laboratory, Report 08362, March 23, 1962.
Materials. -- Illinois Type I-11 asphalt concrete.

1. Limestone - Material Service Company, Chicago, Ill.
2. Torpedo sand - Material Service Company, Chicago, Ill.
3. Lake sand - Material Service Company, Chicago, Ill.
4. Limestone dust - Waukesha Lime and Stone Co., Waukesha, Wis.
5. Asbestos grade 7M06 - Johns-Manville Products Corporation, Manville, N. J.
6. Asphalt A. C. 85/100 - Trumbull Asphalt Company, Detroit, Mich.

The asbestos mixture was identical to the regular mix with the exception that 2.5 percent of the limestone dust was replaced with an equal amount of 7M06 asbestos fiber; and the asphalt content was increased from 5.8 to 6.8 percent.

Test Method

Immediately after preparation of the mixtures, they were loosely placed in quart friction top cans with the cover removed. After the mixtures had cooled to room temperature, the lids were placed to air-seal the cans which were stored in the laboratory at room temperature.

One set of mixtures was tested for asphalt properties immediately after cooling. The others were tested after 3, 6, 9, and 12 months of aging.

Asphalt extraction using a Houghton centrifugal extractor with trichloroethylene and recovered by the Abson method (ASTM D 1856-61T). Tests included penetration, ductility, and ash content. Results are given in Table 16.

TABLE 16

RESULTS OF AGING TESTS ON BITUMINOUS MIXTURES[1] WITH AND WITHOUT ASBESTOS[2]

Aging Period (mo)	Asphalt (%)		Penetration at 77 F		Ductility at 77 F		Ash (%)	
	No A. C.	2.5% A. C.	No A. C.	2.5% A. C.	No A. C.	2.5% A. C.	No A. C.	2.5% A. C.
Initial	5.7	6.7	75	74	150+	150+	1.5	1.9
3	5.7	6.7	57	60	150+	150+	0.5	1.1
6	5.8	6.8	48	48	150+	150+	1.6	0.8
9	5.8	6.8	46	48	150+	150+	1.4	1.8
12	5.9	6.9	46	48	150+	150+	1.5	0.5

[1]Original asphalt: penetration @ 77 F, 100/5 = 94; ductility @ 77 F, 5/60 = 150+ cm.
[2]Chicago Testing Laboratory, Inc., Report 08362, March 23, 1962.

Appendix C

PLANT MIX AND CORE ANALYSIS AFTER PLACEMENT, CALGARY, ALBERTA-ALYTH FREEWAY

Laboratory: MacDonald and MacDonald, Ltd. - Rep. 5, 30 June, 1960
Order No: Tender 0-119
Part #2-B
Mix Classification: III

Plant Mix Sample Analysis:

Property	Standard Mix			Mix 1[a]			Mix 2[a]		
Specimen No.	23a	23b	23c	25a	25b	25c	26a	26b	26c
Marshall stability (lb)	3,118	2,407	2,407	2,115	2,545	3,120	3,540	2,830	3,400
Marshall flow (1/100 in.)	10	10	9	19	18	12	8	11	9
Asphalt content by extraction (%)		5.52			9.17			6.90	
Sp. gr. cake		2.34			2.35			2.32	
Sp. gr. agg.		2.62			2.62			2.61	
VMA (%)		15.7			17.50			17.20	
Voids filled (%)		82.1			100+			93.40	
Voids air (%)		2.8			None			1.20	
Unit wt (pcf)		146.2			146.6			144.7	

[a]Containing 3 percent 7M06 asbestos.

Core Analysis[1]:

Property	Asbestos Modified (3% 7M06)		
Specimen No.	25a	25b	25c
Marshall stability (lb)	970	1,150	950
Marshall flow (1/100 in.)	21	15	16
Asphalt content by extraction (%)		7.65	
Sp. gr. mix		2.38	
Sp. gr. agg.		2.61	
VMA (%)		15.90	
Voids filled (%)		100+	
Voids air (%)		None	
Unit wt (pcf)		148.5	

[1]Cores taken from pavement laid from loads sampled for preceding plant mix analysis; only average values reported for constituent analysis.

Appendix D

PLANT MIX ANALYSIS, ST. LOUIS PAVEMENTS*

Laboratory: Municipal Testing Laboratory, City of St. Louis.

Samples from Strodtman Avenue, October 6, 1959

Property	Lab. No. 593641		Lab. No. 593642	
Sample No.	1		2	
% asbestos (7M06)	2		3	
Bitumen content	8.4		9.8	
Density (pcf)	150.4		147.9	
Marshall stability (lb)	1,070	1,130	1,083	1,050
Marshall flow (1/100 in.)	29	27	31-41	34-49

*"Note the unusually high flow values obtained with relatively high stability, considering the asphalt content. We noticed that the load on the stability test dropped in the usual manner (yield point) and then picked up again and held the same (optimum) stability for an additional .010-in. to .015-in. of flow. In a regular asphaltic concrete sample the load does not pick up again once it fails. The initial flow value is normally the point of failure."

Report of October 9, 1959, by John M. Wendling, Chief Engineer.

Samples from Manchester Avenue at
Tower Grove Avenue, November 5, 1959

Property	Lab. No. 594078	Lab. No. 594079
% asbestos:		
7M06	1.2	1.2
4T28	0.6	0.6
% bitumen	9.1	9.4
Density (pcf)	151.2	151.4
Marshall stability (lb)	1,428	1,188
Marshall flow (1/100 in.)	32-39	30[a]

[a] Load did not pick up again.

Sample from Manchester Avenue Near
Kings Highway Intersection, November 9, 1959

Property	Value
% asbestos	1
% bitumen	8.1
Density (pcf)	152.2
Marshall stability (lb)	1,308
Marshall flow (1/100 in.)	32-39

Appendix E

SKID RESISTANCE TESTS ON MANVILLE PAVEMENTS SEPTEMBER 1960

Test Equipment

Car — 1958 Chevrolet, six-cylinder, four-door sedan, 4,200-lb weight including driver, observer, and instruments.
Tires — Goodyear Custom Nylon, 750 x 14. Good tread, 28-psi tire pressure.
Instruments — Wagner stopmeter with fifth wheel assembly (Fig. 27).

Location

Manville, N. J., North 13th Street.

Test Conditions

Pavement — flushed with fire hoses continuously.
Speed — 30-mph vehicular speed.

Figure 27. Wagner stopmeter with fifth wheel assembly.

Appendix F

WATER PERMEABILITY TEST PROCEDURE

The apparatus in Figure 28 consists of a graduated cylinder with an opening at the bottom attached through a valve connection to a shallow cup which rests in an inverted position on the pavement. The cylinder, valve, and cup are fixed rigidly together by means of a frame which includes lateral projections for support of weights added to counteract the upward force of water pressure on the apparatus.

The solution used is 95 ml of 75 percent aerosol solution in 5 gal of water (as in the California test method). Dye is added to the solution to facilitate reading the height of the water column during testing.

Initial procedure is as follows:

1. Clean pavement surface with wire brush.
2. Mark a 6-in. diameter circle on pavement with crayon using a template.
3. Apply sealer to circle. Johns-Manville "Albaseal" caulking strips or equivalent.
4. Put the permeability apparatus on the pavement with the periphery of the cup resting on the sealer.
5. Apply pressure to the lateral extensions on the frame and rotate slightly at the same time. Apply finger pressure against sealer on the outside of the cup to insure a tight seal.
6. Put the necessary weights on the frame extensions.
7. Fill the cylinder with solution keeping the valve closed.

To start the test, open the valve and start the stop watch. Normally, 30 sec are required for solution to fill the cup and for air to escape to the surface, during which time solution is added to keep the cylinder filled.

At 30 sec, the initial surface level of the liquid at the top of the cylinder is read using a millimeter scale attached to the cylinder. The decreasing level of the surface is observed and recorded at 15-sec intervals for a total of 4 min. By calibration of the cylinder, the amount of solution penetrating into the pavement is obtained and plotted against time. The slope of the curve from 30 to 90 sec is arbitrarily used as a measure of surface permeability. Readings taken from 90 sec to 4 min are used primarily for very tight pavements to insure that the flow is continuous and to increase precision of flow rates obtained.

For pavements with a very low permeability, the sensitivity of flow rates is increased by extending the initial height of the water column, using a small diameter graduated tube on top of the plastic cylinder (Fig. 28). This method was used to obtain all of the permeability data described in this report.

The relatively small decrease in head during each test period was found to have a negligible effect on flow rates. The effect can be measured for any pavement by duplicating tests at a lower head.

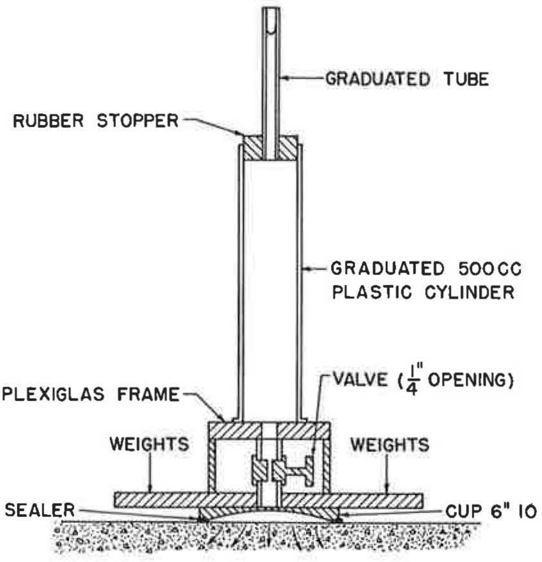

Figure 28. Cross-section of water permeability apparatus.

Appendix G

RECENT ASBESTOS-ASPHALT CONCRETE SURFACE COURSES
WITH SUBSTANTIALLY INCREASED ASPHALT CONTENT

City, State, Street Name or Rt. No.	Supervisory Authority	Date of Installation	Type of Installation	Type or Classification Mix	% Asbestos	Type of Asphalt Pen.	Optimum % A.C. for Standard Mix	% A.C. in Asbestos Modified Mix	% Increase in A.C. Content	Remarks
*Ottawa, Ill.	Ill. Hwy. Dept.	9/62	1½-in. wearing course	A-1 IV-A Ill. I-11	2.0	85/100	5.5	6.7	22	Rehabilitated AASHO Test Road
*Hampden/Newburg, Me.	Me. Hwy. Dept.	6/62	1½-in. wearing course resurfacing over existing bit. concrete pavement	A-1 IV-A	1.5	85/100 60/70 120/150	6.4 6.4 6.4	7.8 7.8 7.8	22 22 22	3 pen. grades of asphalt being evaluated in combination with fillers
*Raleigh, N. C., Rt. 64	N. C. Hwy. Dept.	5/23/62	1½-in. wearing course on 3½-in. resurfacing project		2.5 1.67		6.7 6.7	8.5 8.0	27 20	
St. Louis, Mo., Jefferson Ave.	City of St. Louis	8/6/62	3/4-in. thin surfacing over spalled p.c. concrete	A-1 V-A	1.5	60/70	5.8	8.5	46	Thin overlay
Madawaska, Me., US 1 Supr. to Canada	Me. Hwy. Dept.	7/11/61	1-in. surface course over asbestos-asphalt concrete binder and base course	A-1 V-A Modified	2.0	85/100	7.0 (est.)	8.5 to 9.1	22 to 30	New construction
*Asbestos/Danville, Quebec, Canada Rt. 12	Que. Dept. Hwys	6/28/62	1-in. wearing course	A-1 IV-A	2.0	85/100	6.0	8.0	33	New construction
*Lafayette, Ind., US 52 Bypass, Ind. 25	Ind. Hwy. Dept.	7/25/52 7/25/62	7/8-in. surface course 7/8-in. surface course	Ind. Type-B A-1 Skip A	2.0 2.0	60/70 60/70	6.5 6.5	8.2 8.2	26 26	
*Last Chance, Colo., H.S. 36	Colo. Hwy. Dept.	4/4/62	1-in. wearing surface on 1½-in. binder on 4-in. soil cement base (new construction)	Pit run Plant mix	2.0	85/100	5.8	7.3	26	New construction
Banff, Alberta Trans-Canada Highway	Canadian Dept. of Public Work	6/5/52	-	-	-	-	5.3	7.5	42	New construction

*Experimental project.

Performance of Asphalt Pavements Subjected to De-Icing Salts

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This paper describes a laboratory investigation on the performance of asphalt pavement subjected to the common de-icing salts. Various tests were made on the specimens immediately after they were prepared and after 10, 50 and 100 daily test cycles. Comparisons of the test results indicated that the repetitive daily applications of the de-icing salts to melt ice from the surfaces of specimens had no significant effects on specimen stabilities, or on the penetration, softening point, or ductility of asphalt recovered from the specimens. No loss of aggregate from any specimens occurred during the tests.

No detrimental effects of the de-icing salts on the asphalt pavement specimens subjected to the testing conditions used for the investigation were indicated by the test results. The study offers evidence that well-designed and well-constructed asphalt pavements are not damaged by sodium and calcium chloride salts used for ice and snow control.

•INCREASED ATTENTION to the removal of snow and ice from pavements has resulted from the growing demand for clean, bare pavements during the winter. The use of chemicals and abrasives for snow and ice control has increased. The widespread use of sodium and calcium chloride salts for this purpose has also caused growing concern about the effects of these common de-icing salts on pavements.

The performance of asphalt pavements subjected to these salts was studied in a laboratory investigation conducted by the Asphalt Institute. Asphalt pavement specimens were subjected to repetitive daily test cycles simulating the field conditions existing when de-icing salts are applied. A mixture of sodium and calcium chloride salts was used to melt ice frozen on the surfaces of pavement specimens during the test cycles. Similar pavement specimens, which were salt-free, were subjected to freezing and thawing conditions and to water, freezing and thawing, during the daily test cycles. Tests were made on the specimens at various intervals to measure stability, loss of aggregate, and properties of the asphalt extracted and recovered from the specimens. These test values, and comparisons of the test values for specimens exposed and not exposed to the de-icing salts, indicated the performance of the specimens to which the de-icing salts had been applied.

PREPARATION OF TEST SPECIMENS

An Asphalt Institute Type VIII-a sheet asphalt and two Asphalt Institute Type IV-b dense-graded asphalt concretes, each containing different aggregates, were used in the investigation. Replicate specimens of each paving mixture were prepared in accordance with procedures specified by the Marshall test, ASTM Designation D 1559. Fifty compaction blows were applied to each end of the specimens. The specimens were made

Paper sponsored by Committee on Effects of Natural Elements and Chemicals on Bitumen-Aggregate Combinations and Methods for Their Evaluations.

TABLE 1

MIX COMPOSITIONS

U. S. Standard Sieve Size	Percent Passing		
	Sheet Asphalt Md. Sand Agg.[1]	Asphalt Concrete	
		N. Y. Trap Rock Agg.[2]	Mich. Gravel Agg.[3]
½-in.	--	100	100
⅜-in.	--	80	80
No. 4	--	60	60
No. 8	100	42	42
No. 30	95	23	26
No. 50	45	17	13
No. 100	20	12	8
No. 200	15	8	7
% asphalt, total mixture	11.0	6.3	5.0

[1] Composite contained 89.1 percent Maryland sand, and 10.9 percent limestone dust mineral filler.
[2] Entirely New York trap rock.
[3] Composite contained 72 percent Michigan gravel, 22 percent Maryland sand, and 6.0 percent limestone dust mineral filler.

with optimum asphalt contents determined by the Marshall method of mix design (1). Composition of the test specimens is given in Table 1. Table 2 gives test properties of the 85-100 penetration grade asphalt used for preparation of all specimens.

TABLE 2

ASPHALT CEMENT TEST PROPERTIES

Property	Value
Penetration, 77 F, 100 g, 5 sec	90
Viscosity at 275 F, (centistokes)	291
Penetration after thin film oven test, (% of original)	58
Ductility at 77 F (cm)	150+
Softening point, ring and ball, (°F)	118
Specific gravity, 77/77 F	1.022

DESCRIPTION OF TESTS

Replicate specimens of each mix were divided into three groups. All groups were subjected to a daily cycle of freezing at 20 F and thawing at 50 F. Specimens in Group I were subjected only to the daily freezing and thawing cycles and were not contacted by water or de-icing salts. Forty milliliters of water was placed on the upper surfaces of specimens in Group II, and the water allowed to freeze and thaw during each daily test cycle. De-icing salts were not applied to specimens in Group II. Forty milliliters of water was placed on the surfaces of specimens in Group III. A mixture of 5.4 g of crushed rock salt and 0.6 g of flake calcium chloride was applied to melt the ice frozen on the surfaces of specimens in the third group during each daily test cycle. The specimens in Groups II and III were fitted with watertight rubber collars to contain the 0.2 in. depth of water or salt solution on their upper surfaces. The rubber collars were cemented to the sides of the specimens with approximately 3.5 g of the same asphalt used in the specimens.

A program-controlled temperature cabinet, providing temperature control within ±2 F, was used to obtain repetitive daily freezing and thawing cycles. The specimens remained in the cabinet during the entire investigation, except for the short time each day when the water or the salt solution was poured from the surfaces of specimens in the second and third groups. The test specimens in the temperature cabinet are shown in Figure 1. Appendix A gives the detailed testing conditions used for the daily test cycles. In addition to the main series of specimens used for the daily test cycles, three specimens of each mix were stored on a laboratory shelf at room temperature and tested at the conclusion of the investigation.

Various tests were made on the specimens immediately after they were prepared, and after 10, 50, and 100 daily test cycles. Tests included the following:

1. Marshall stability and flow (ASTM Method D 1559).
2. Extraction and recovery of asphalt (ASTM Method D 762).
3. Penetration (ASTM Method D 5), softening point (ASTM Method D 36), and ductility (ASTM Method D 113) of the recovered asphalt.

The Marshall test was selected to measure stability, largely because the test is quite sensitive to asphalt binder viscosity changes and density changes of specimens.

Figure 1. Test specimens in test cabinet.

The penetration, softening point, and ductility tests were used to obtain direct measurements of changes in the asphalt extracted and recovered from the specimens during the investigation. Specimen weights were also recorded during the tests to provide an indication of aggregate losses. The surface-dry specimens were weighed to the nearest 0.1 g immediately after preparation and after 10 and 100 daily test cycles.

Various combinations and different application rates of sodium and calcium chloride are used for ice and snow control. The salt application rate for the laboratory tests was equivalent to 1.36 lb of salt mixture per square yard. At a temperature of 20 F, it was sufficient to melt the ice from the surfaces of specimens within 1½ hr. The salt application rate in the laboratory tests was higher than that normally used in the field. It was selected to produce a severe laboratory test, recognizing that under some field conditions high concentration salt solutions might come in contact with pavements.

TEST RESULTS

The paving mixture specimens used for the tests were representative of varying stability pavements made with typical aggregates. Marshall stability and flow values for specimens tested immediately after preparation are given in Table 3. The Maryland sand aggregate sheet asphalt had a Marshall stability of 618 lb and flow value of 16. The Michigan gravel aggregate asphalt concrete had a Marshall stability of 1,125 lb and flow value of 12, and the New York trap rock aggregate asphalt concrete had a Marshall stability of 2,093 lb and flow value of 15.

It was predicted that the Marshall stability test probably would not reflect even a slight amount of scaling or aggregate loss from the surface of the specimens. Marshall stability would, however, be greatly reduced due to severe deterioration of specimens and would also decrease due to increasing volume or decreasing density of specimens. Marshall stability would increase due to appreciable increases in the viscosity or decreases in penetration of the asphalt in specimens. For this investigation, changes in the Marshall stability of the specimens are of most interest as a direct measure of any severe deterioration, and as an indirect indication of density changes or appreciable asphalt consistency changes.

TABLE 3

TEST PROPERTIES[1] OF SPECIMENS TESTED IMMEDIATELY AND AFTER AGING IN AIR DURING ENTIRE STUDY

Specimen	Property	Sheet Asphalt Maryland Sand Agg.	Asphalt Concrete	
			N. Y. Trap Rock Agg.	Michigan Gravel Agg.
Tested immediately	Air voids (%)	6.0	4.1	3.4
	Stability (lb)	618	2,093	1,125
	Flow (0.01 in.)	16	15	12
	Pen. at 77 F (0.1 mm)	85	80	79
	S. P. (°F)	119.0	119.0	118.0
	Duct. at 77 F (cm)	150+	150+	150+
Aged in air	Air voids (%)	5.9	4.0	3.4
	Stability (lb)	667	2,150	1,247
	Flow (0.01 in.)	16	16	11
	Wt. gain (%)	0.03	0.16	0.02
	Pen. at 77 F (0.1 mm)	65	66	62
	S. P. (°F)	122.5	122.5	123.0
	Duct. at 77 F (cm)	150+	150+	150+

[1]Values are averages for three test specimens; penetration, softening point, and ductility test values are for extracted and recovered asphalt from duplicate test specimens.

TABLE 4

TEST PROPERTIES[1] OF SPECIMENS SUBJECTED TO DE-ICING SALT TESTS

Group	Property	Sheet Asphalt Maryland Sand Agg.			Asphalt Concrete					
					N. Y. Trap Rock Agg.			Michigan Gravel Agg.		
		10 Cycles	50 Cycles	100 Cycles	10 Cycles	50 Cycles	100 Cycles	10 Cycles	50 Cycles	100 Cycles
I	Air voids[2] (%)	6.0	5.9	5.9	4.0	4.1	4.3	3.3	3.3	3.2
	Stability (lb)	613	655	620	1,950	2,067	1,955	1,145	1,265	1,265
	Flow (0.01 in.)	15	16	15	14	15	14	12	11	11
	Wt. gain (%)	0.04	--	0.06	0.12	--	0.22	0.01	--	0.01
	Pen. at 77 F (0.1 mm)	72	74	72	74	75	71	73	73	70
	S. P. (°F)	120.5	121.0	121.5	122.0	121.0	121.5	120.5	120.5	121.5
	Duct. at 77 F (cm)	150+	150+	150+	150+	150+	150+	150+	150+	150+
II	Air voids[2] (%)	6.0	6.1	6.2	4.1	4.0	4.5	3.4	3.5	3.5
	Stability (lb)	635	638	588	2,040	2,178	1,972	1,335	1,175	1,108
	Flow (0.01 in.)	15	15	16	16	15	15	12	11	12
	Wt. gain (%)	0.04	--	0.03	0.07	--	0.12	0.02	--	0.03
	Pen. at 77 F (0.1 mm)	72	75	74	72	73	71	72	71	70
	S. P. (°F)	120.0	121.0	120.0	122.5	121.5	121.5	121.0	121.0	122.0
	Duct. at 77 F (cm)	150+	150+	150+	150+	150+	150+	150+	150+	150+
III	Air voids[2] (%)	6.2	6.0	6.3	4.1	4.3	4.2	3.3	3.4	3.4
	Stability (lb)	585	628	613	1,968	1,910	1,938	1,170	1,263	1,170
	Flow (0.01 in.)	15	16	15	14	13	14	12	12	11
	Wt. gain (%)	0.04	--	0.04	0.10	--	0.16	0.02	--	0.02
	Pen. at 77 F (0.1 mm)	74	74	74	73	75	76	71	74	70
	S. P. (°F)	120.0	121.5	120.0	120.0	120.5	121.0	121.0	120.5	122.0
	Duct. at 77 F (cm)	150+	150+	150+	150+	150+	150+	150+	150+	150+

[1]Values are averages for duplicate test specimens; penetration, softening point, and ductility test values are for extracted and recovered asphalt from duplicate test specimens.
[2]Determined before specimens were subjected to de-icing salt tests.

Marshall stabilities for the sheet asphalt and the two asphalt concretes tested immediately and after 10, 50, and 100 daily test cycles are given in Table 4 and shown in Figure 2. There were no significant changes in the stability of any of the mixes subjected to as many as 100 applications of the de-icing salts to melt ice frozen on the surfaces of the specimens. There also were no significant differences for any of the mixes between stabilities of specimens exposed to the de-icing salts and those subjected only to freezing and thawing, and to freezing and thawing water only. The stability test results indicate no severe deterioration of specimens occurred due to exposure to the de-icing salts or to the other testing conditions. They also indicate that no appreciable

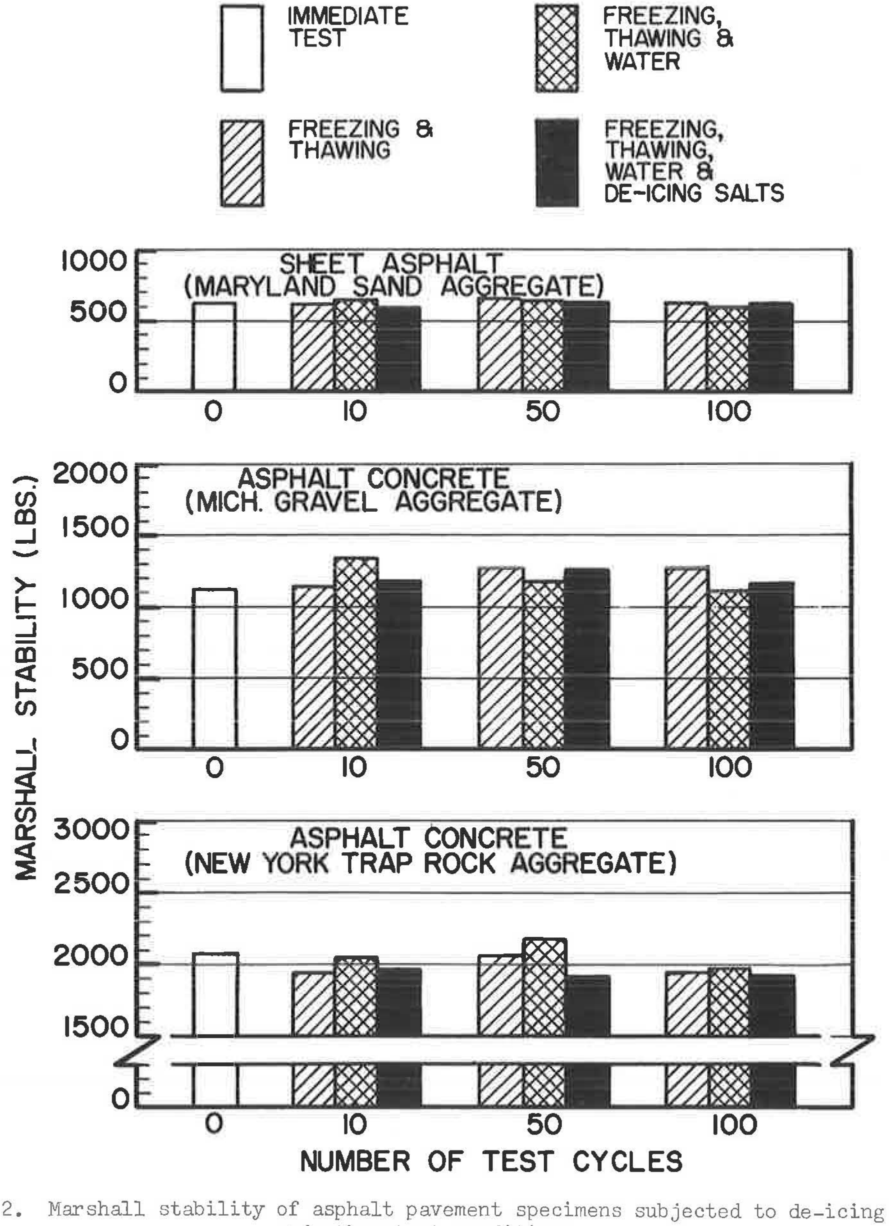

Figure 2. Marshall stability of asphalt pavement specimens subjected to de-icing salts and other test conditions.

density or volume change occurred in the specimens, and that no large changes occurred in the consistency of the asphalt in any of the specimens due to exposure to the de-icing salts.

Increases in Marshall flow values would be expected due to volume increases in specimens or severe deterioration of specimens. Marshall flow values for sheet asphalt and the two asphalt concretes tested immediately, and then after 10, 50, and 100 daily test cycles, are given in Table 4. They are also shown in Figure 3. Like the stabilities, flow values for the specimens were not affected by the de-icing salts or other testing conditions. The flow values did not change significantly during the 100 daily test cycles for any of the mixes.

Marshall stability and flow value test results for specimens stored in the laboratory at room temperature and tested at the conclusion of the investigation are given in Table 3. There were no significant differences between the Marshall stability and flow values for specimens aged at room temperature and those subjected to the de-icing salts and other testing conditions for 100 daily test cycles.

The asphalt coating the aggregate particles was the paving mixture component that had the most direct contact with water and the salt solutions during the tests. The effect of the de-icing salt solutions on the asphalt is indicated by comparison of the test properties of recovered asphalt from specimens contacted and not contacted by the de-icing salts. Recovered asphalt test properties are given in Table 4.

Penetration test values for asphalt extracted and recovered from sheet asphalt and the asphalt concrete specimens tested immediately, and after 10, 50, and 100 daily test cycles, are shown in Figure 4. Penetrations of the recovered asphalt decreased 5 to 11 points for the different paving mixtures during mixing and compaction of specimens. Further decreases of 6 to 13 penetration points occurred during the first 10 test cycles. Thereafter, penetrations did not change significantly for any of the mixes up to 100 daily test cycles. Comparison of penetrations of recovered asphalt from specimens contacted and not contacted by the de-icing salts indicates that the consistency of the asphalt as measured by the penetration test was not affected by the de-icing salts for any of the mixes.

Softening points for extracted and recovered asphalt from sheet asphalt and asphalt concrete specimens determined immediately after the specimens were prepared, and after 10, 50, and 100 daily test cycles, are shown in Figure 5. Trends in recovered asphalt consistency measured by the softening point tests agreed with consistency trends measured by the penetration tests. Increases in softening points during mixing and compaction, and a further increase during the first 10 daily test cycles, occurred. Thereafter, softening points remained approximately the same. Softening points were not significantly different for the different testing conditions, indicating that consistency of asphalt in the specimens as determined by the softening point test was not affected by the de-icing salts applied to any of the mixes.

The ductility of the original asphalt was greater than 150 cm. The ductilities of asphalt recovered from all specimens immediately after their preparation, and after 10, 50, and 100 daily test cycles for all the testing conditions, were also greater than 150 cm. The de-icing salts caused no changes in the ductilities of recovered asphalt, within the testing limit of the 150-cm ductility apparatus used.

Penetrations, softening points, and ductilities of asphalt recovered from specimens aged at room temperature during the investigation are given in Table 3. Penetrations were slightly lower and softening points were slightly higher for asphalt recovered from these specimens than for the asphalt recovered from the 100 test cycle specimens for all mixes. Apparently the higher temperature and more exposure to light caused slightly greater hardening of the asphalt in specimens aged at room temperature. Ductilities of asphalt recovered from the specimens aged at room temperature during the investigation were greater than 150 cm, the same as ductilities of asphalt recovered from all specimens after 100 daily test cycles.

The results of tests on asphalt recovered from specimens during the investigation indicated that the asphalt in the specimens was not significantly affected by exposure to the de-icing salts.

Any appreciable loss of aggregate or scaling due to the de-icing salts or other testing conditions would be indicated by a decrease in weight of the specimens. Weight

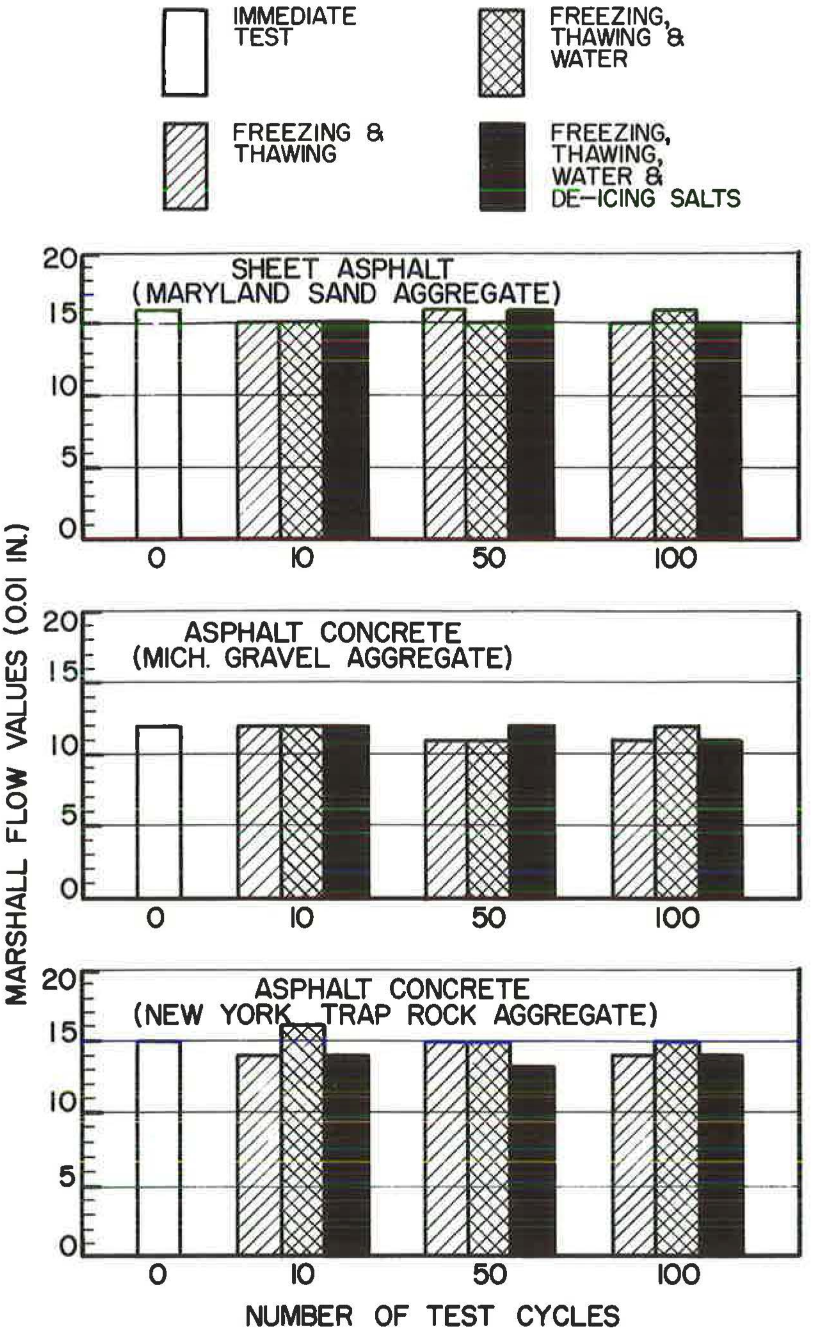

Figure 3. Marshall flow values of asphalt pavement specimens subjected to de-icing salts and other test conditions.

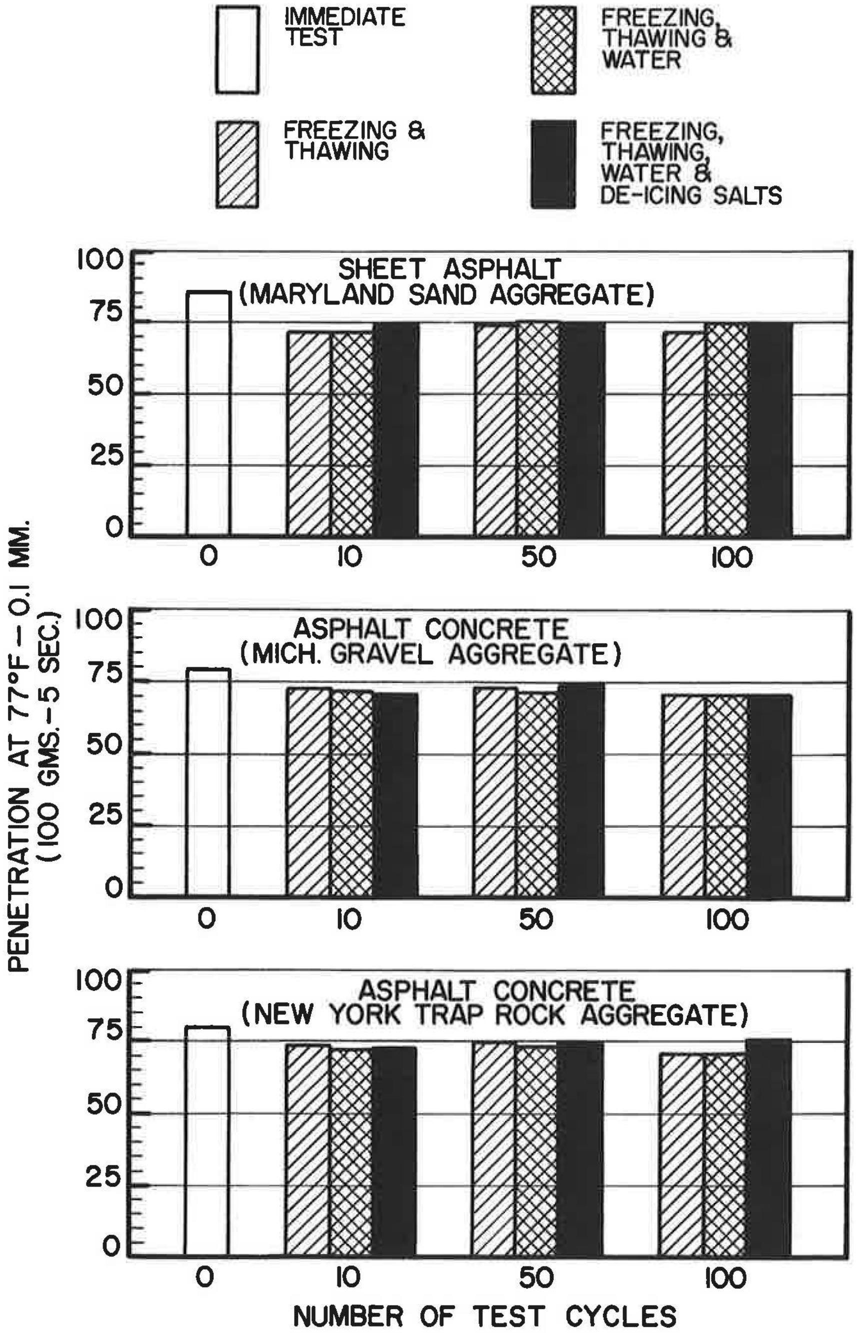

Figure 4. Penetration of asphalt recovered from pavement specimens subjected to de-icing salts and other test conditions.

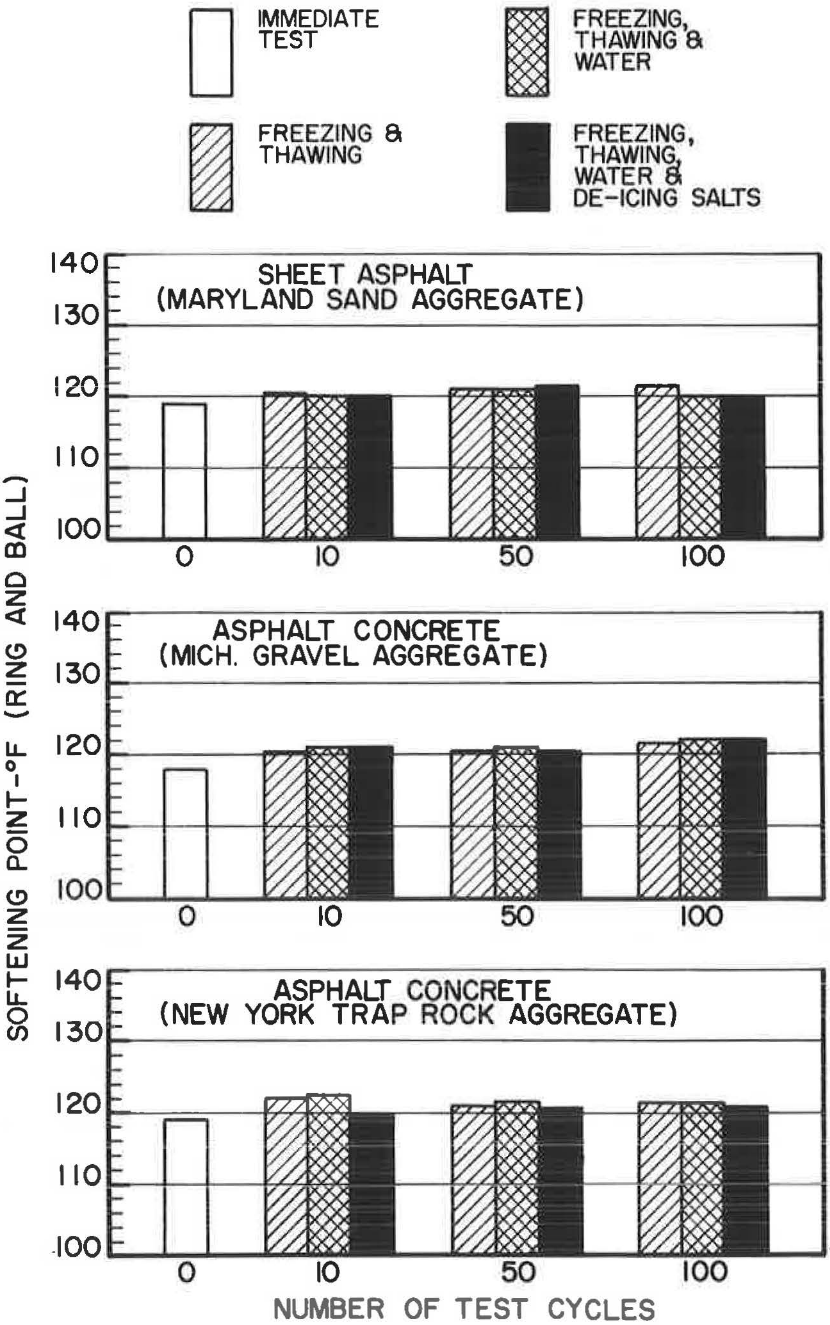

Figure 5. Softening point of asphalt recovered from pavement specimens subjected to de-icing salts and other test conditions.

changes of surface-dry specimens after 10 and 100 daily test cycles, expressed as percent of the original specimen weight, are given in Table 3. They are shown in Figure 6 for the different mixes. All specimens, for all testing conditions, and including the specimens aged at room temperature, gained in weight during the investigation. The weight gains, due to increase in moisture content of the surface-dry specimens, were very small for sheet asphalt and gravel aggregate asphalt concrete. They were only slightly higher for trap rock aggregate asphalt concrete. There were no appreciable differences in weight gains of specimens subjected to the different test conditions. It is evident from these results that no loss of aggregate occurred due to specimen exposure to the de-icing salts. The small weight gains were due to the normal increase in the moisture contents of the specimens exposed either to water or the atmosphere. No loose particles or losses of aggregate from the surfaces of specimens were observed when the water or salt solutions were poured from the surfaces of specimens during the investigation.

Differences became noticeable in the surface appearance of specimens subjected to the different testing conditions at about 50 daily test cycles. The surfaces of all specimens exposed to water and to the de-icing salts became slightly lighter in color. There was little difference, if any, between the appearance of specimens subjected to these two test conditions. The surface appearance of these specimens was quite similar to the surface appearance of asphalt pavements exposed to outdoor weathering. The surfaces of all specimens were photographed after 10, 50, and 100 daily test cycles. Michigan gravel aggregate asphalt concrete specimens, typical of the appearance of the other mixes, are shown in Figures 7, 8, and 9 after 10, 50, and 100 daily test cycles, respectively. The numbers under the specimens designate the testing condition. Freezing and thawing only are designated by the number 1. Freezing, thawing, and water are designated by the number 2; and freezing, thawing, water and de-icing salts are designated by the number 3. The slight changes in the appearance of the asphalt-coated aggregate particles on the surfaces of specimens contacted by water and by the de-icing salts were not reflected in the test properties of asphalt recovered from the specimens. Because no losses of aggregate particles occurred, the slight changes in surface appearance of specimens were not associated with retention of aggregate particles on the surfaces of specimens by the asphalt binder.

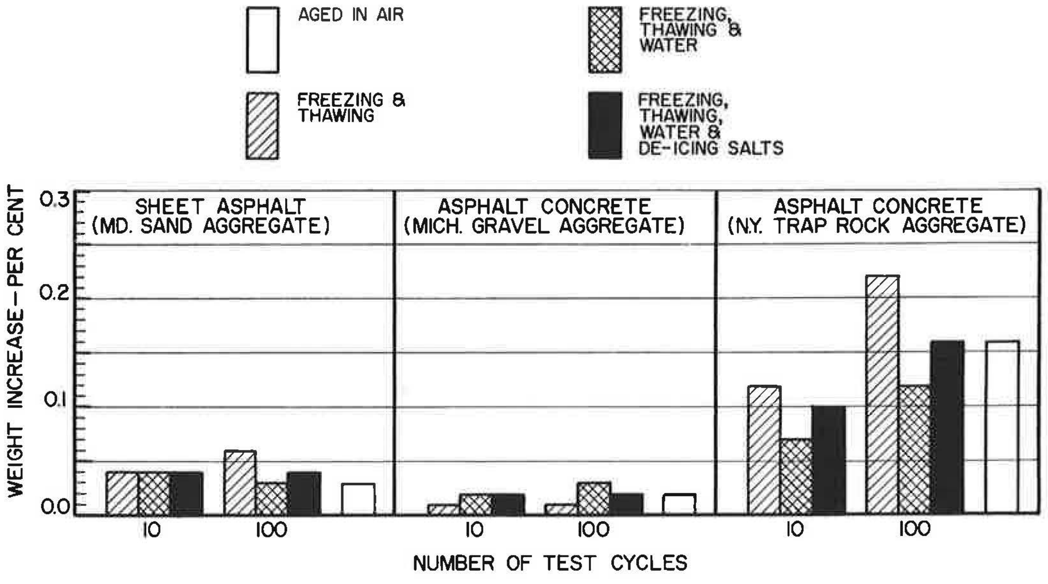

Figure 6. Weight changes of asphalt pavement specimens subjected to de-icing salts and other test conditions.

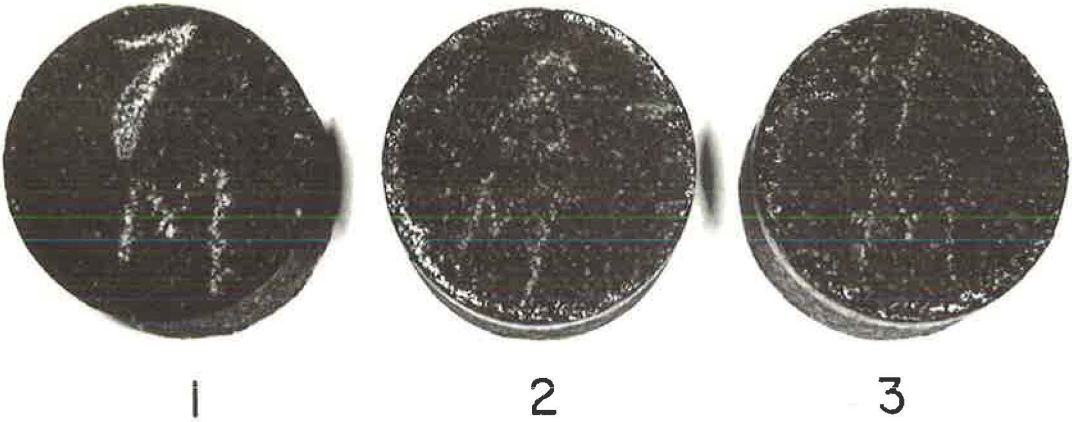

Figure 7. Asphalt concrete samples after 10 cycles.

Figure 8. Asphalt concrete samples after 50 cycles.

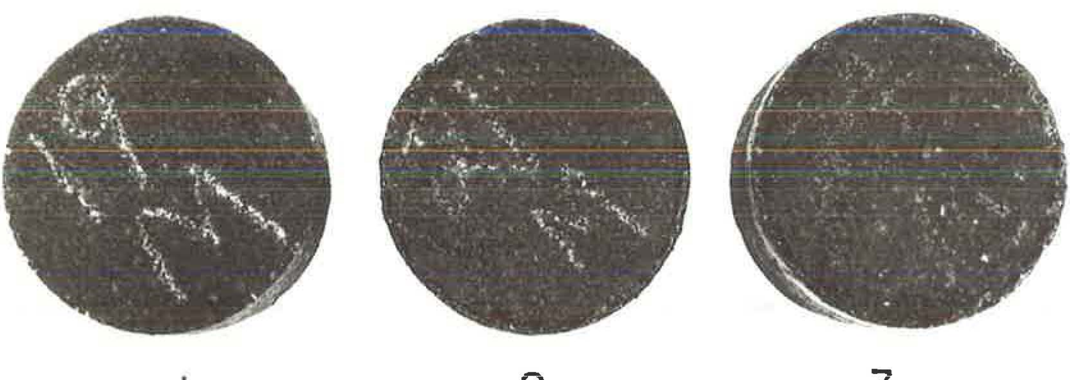

Figure 9. Asphalt concrete samples after 100 cycles.

SUMMARY AND CONCLUSIONS

The investigation described in this paper was made to study the performance of asphalt pavements subjected to the common de-icing salts. Typical sheet asphalt and dense-graded asphalt concrete paving mixture specimens were subjected to daily test cycles simulating field conditions existing when de-icing salts are used on pavements.

It was found that as many as 100 daily applications of a mixture of sodium chloride and calcium chloride salts to melt ice from the surfaces of pavement specimens had no significant effect on the specimens. The stabilities of the specimens were not affected, and no loss of aggregate or scaling occurred. Test properties of asphalt recovered from the specimens were not affected by the de-icing salts. The results of the laboratory tests indicate that the performance of similar well-designed and well-constructed dense-graded aggregate asphalt concrete and sheet asphalt pavements would not be affected by repeated applications of sodium chloride and calcium chloride salts for winter snow and ice control.

REFERENCE

1. "Mix Design Methods for Hot-Mix Asphalt Paving." Asphalt Institute, Manual Series 2.

Appendix

DAILY TEST CYCLES

All test specimens in Groups I, II, and III were placed in the temperature control cabinet, maintained at a temperature of 20 F, the evening before the tests began. The daily 24-hr test cycles started the following day at 9:00 AM. Testing conditions and the procedures for handling the specimens during each daily test cycle were as follows:

1. At 9:00 AM, with the cabinet temperature at 20 F, 40 ml of water was placed on the upper surfaces of specimens in Groups II and III.
2. At 11:30 AM, with the cabinet temperature at 20 F, a mixture of 5.4 g of crushed rock salt and 0.6 g of flake calcium chloride was placed on the ice frozen on surfaces of specimens in Group III.
3. At 1:00 PM, the cabinet temperature was increased at a rate of approximately 0.67 F per min until, at 1:45 PM, the cabinet reached and maintained a temperature of 50 F.
4. At 3:00 PM, specimens in Groups II and III were rapidly removed from the cabinet and the water or salt solutions poured from their surfaces. The specimens were replaced in the cabinet maintained at a temperature of 50 F.
5. At 9:00 PM, the cabinet temperature was decreased at the rate of approximately 0.67 F per min, until at 9:45 PM, a temperature of 20 F was reached. The 20 F temperature was maintained during the completion of the daily test cycle until 9:00 AM the following day.
6. The specimens were left in the cabinet maintained at a temperature of 20 F during weekends and holidays.

Emulsified Petroleum Oils and Resins in Reconstituting Asphalts in Pavements

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Asphalts in paving mixtures change in composition, or age, with time. The rate and degree of change depends on their chemical composition, environmental conditions, and length of exposure.

In the case of hot-plant mixes, the aging process starts even before pavement construction because of exposure to air at high temperatures in thin films during the hot-mix cycle. Sooner or later the asphalt will reach a state of brittleness which manifests itself in the form of pitting and raveling of the surface or shrinkage and brittleness cracking, with eventual spalling.

Addition of suitable asphalt components at the appropriate time during the aging process can stop aging by reconstitution of the weathered asphalts. Appropriate treatment of pavements returns the weathered asphalt to its original condition. In many instances, the properties of the asphalts are so improved that they surpass those in the asphalt prior to mixing with the aggregate.

Laboratory data illustrate the manner of aging of asphalts and the concept of restoring properties by use of selected fractions of petroleum oils and resins. Reasons are cited for the selection of an emulsion system for bringing together the aged asphalt and the particular oils and resins.

The engineering properties of the preparation required for obtaining the desired results are described, together with a summary of 3 years' field experience. Test data from pavements subjected to treatment under various conditions are also presented, including evaluations and measurements of such physical properties of the pavement as permeability, texture, stability, and skid resistance.

• IT IS characteristic of asphalts, in asphalt paving mixtures of all types, to change in composition with time. After construction and during service on the road, this process (referred to as aging in this report) is generally a gradual one with the rate and degree of change dependent on the original chemical composition of the asphalt, its environmental conditions, and length of exposure to weathering. In the case of hot-plant mixes, the aging process starts even before pavement construction because the asphalts are exposed in thin films to air at high temperatures during the hot-mix cycle. The rate of change is often influenced by the catalytic action of the aggregate surface on which the asphalt film is deposited.

Many studies on both the causes and effects of asphalt aging have been reported in the literature (1, 2, 3) with general agreement on the nature of the problem but with some differences of opinion on the mechanics of the aging process within the asphalt itself. The interpretations range from the assumption that asphalt hardening or brittleness is purely an evaporation phenomenon to the more complex explanations predicated

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on the correlation of asphalt composition, determined by fractional chemical analysis, to long-term performance on the road. The fact remains, however, that sooner or later the asphalt will reach a state of brittleness which is manifested in the form of pitting and raveling of the surface, or shrinkage and brittleness cracking, or spalling, or combinations thereof, with eventual deterioration of the pavement (4, 5).

It would be of considerable advantage, therefore, to asphalt users if it were possible to retard, stop, or reverse the aging of asphalt, in situ, by the addition of suitable asphalt components at some appropriate time during the aging process. Such a treatment would require a material that would effectively rejuvenate the aged asphalt with minimum interference with the serviceability of the pavement, and yet be economically feasible. This paper describes the results obtained with a material especially formulated as such a reclaiming agent by combining with and reconstituting aged asphalt, in situ, thereby restoring its plasticity.

FUNDAMENTALS OF COMPOSITION OF RECLAIMING AGENT

Disregarding the details and minor differences of opinion regarding the chemical composition of asphalts, it is generally accepted that asphalt consists of two main fractions, asphaltenes (insoluble in pentane) and maltenes (soluble in pentane), and that the maltenes consist of subfractions of various oils and resins. The Rostler analysis (3) determines the following four principal fractions of maltenes: nitrogen bases (N), first acidaffins (A_1), second acidaffins (A_2), and paraffins (P). The influence of maltenes on the durability of asphalts as cementing agents has been shown to depend on the ratio of the four fractions of the maltenes. The parameter $(N + A_1) / (P + A_2)$, expressive of the ratio of the more reactive to the less reactive fractions, is a useful guide for predicting performance of an asphalt as a binder (3).

The reclaiming agent used in the present work is an emulsion of maltenes designed to change the resins/oils ratio (expressed by the preceding parameter) in the asphalt to increase durability. It is a cationic emulsion of specific properties tailored to facilitate and assure the desired incorporation of the added maltenes.

THE CATIONIC OIL-RESIN EMULSION

A number of products could be designed to serve as an asphalt replasticizer. The reclaiming agent used in the work described in this paper is the product "Reclamite" developed by the Golden Bear Oil Co. Reclamite is described in detail in pending patent applications. The principal general requirements for this type preparation are as follows:

1. Facilitate penetration of the emulsion into the asphalt pavement surface.
2. Effect preferential wetting of the asphalt by the added maltenes.
3. Accelerate penetration of the emulsion into the interstices of the asphalt structure itself.

TABLE 1

SIGNIFICANT PROPERTIES OF RECLAIMING AGENT

Intrinsic properties of maltenes emulsion:	
Avg. particle size, 1:4 dilution (μ)	1.8
Polarity, by electrophoresis	Cationic
Stability, in 1:7 dilution	--[1]
Maltenes content (%)	60 ± 3
Typical properties of maltenes:	
Specific gravity	0.95
Viscosity at 90° C (cP)	25
Flash point, COC (°F)	400
Loss on heating, 3 hr at 325° F (%)	1

[1] No irreversible stratification; no coagulation on contact with sea water.

The essential function of the emulsion after penetration into an asphalt pavement is to deposit the oil and resin components on the films of aged asphalt without disturbing the existing structure of the asphalt-aggregate mix with respect to adhesion, cohesion or stability. Subsequently, the deposited oils and resins flux with and replasticize the aged asphalt and "reclaim" or "rejuvenate" it. The degree of improvement by such treatment depends on the asphalt's state of embrittlement and the composition and quantity of the maltenes comprising the oil-resin blend. Proper design of the reclaiming agent can make the replasticized asphalt more chemically stable than the original because the added components are more durable and in better balance than those generally encountered in asphalts. Table 1 summarizes the significant properties of the product used.

SELECTION OF EMULSION SYSTEM

Laboratory tests were conducted to determine the best manner of adding the reclaiming material. Ease of handling and application were significant factors in deciding that the product should be in the form of an emulsion. However, the most important factor favoring emulsification was the need for a preparation which would penetrate rapidly into the pores of the asphalt paving, without displacing the asphalt films from the aggregate or destroying the existing structure of the asphalt-aggregate mixture.

Table 2 summarizes test results obtained on uniformly prepared laboratory specimens to determine both the rate and depth of penetration of the maltenes applied in various forms, together with test results on kerosene, MC-70 liquid asphalt, and tap water for comparative purposes. The specimens were briquets, 2.5 in. in diameter, 1.6 in. high, with a 2-in. diameter, 0.318-in. deep reservoir pressed into the top side. The briquet mixture consisted of 90 parts by weight of 20-30 mesh Ottawa sand (ASTM C-109), 10 parts portland cement and 7.5 parts asphalt of 48 penetration. Appendix A is the standard procedure used to blend, mix and mold the briquets.

It is evident (Table 2) that a relatively rapid rate of penetration was obtained with all forms of emulsion systems, except the asphalt emulsion (grade SS-1, diluted 2:1 with water) which did not penetrate but formed an asphalt skin at the surface when the water phase evaporated. When applied directly, without a carrier, the oils and resins penetrated very slowly over a 24-hr period. However, the oils and resins, when dissolved in kerosene, also penetrated rapidly.

The briquets were relatively impervious to plain tap water, but with the wetting agent added water penetrated readily. The water with wetting agent even penetrated the treated briquets, but at a much slower rate. This serves as a measure of the relative effect on pavement permeability of these various treatments. The last column (Table 2) indicates that all emulsion systems tested are more effective in decreasing the permeability of an asphalt-aggregate mixture than are the kerosene cutbacks. Moreover, although the SS-1 asphalt emulsion formed a membrane, water still managed to permeate through.

Figures 1 through 12 show the briquets in ultraviolet light used to determine the penetration depth of the fluorescent oils and resins. The procedure was to split one of duplicate briquets and measure depth of penetration to the nearest 0.01 in. As is evident from Table 2, the average depth of penetration for the maltenes in any form of application was virtually the same. The only anomaly was the anionic emulsion that ran through the briquet and out the bottom. This was anticipated, however, because of the repellent nature of the asphalt-coated surfaces to negatively charged particles.

An improvised load-penetration test was run on the same type of briquet to evaluate the effect of the cationic emulsion of maltenes on the cohesiveness of asphalt as compared to other dilution systems and materials. An apparatus applied load to the treated end of the briquet through a 1.128-in. diameter (1.0 sq in.) piston at a strain rate of 0.25 in. per min to a total deformation of 0.250 in. The data obtained from this test are relative but valid for the series of briquets tested. The apparatus and the test are described in Appendix B. Resulting load-deformation diagrams are shown in Figure 13. A refinement of this test is under development.

To establish reference points, two load-deformation curves are shown for freshly-mixed briquets made with 7½ percent by weight of 48-penetration and 200- to

TABLE 2

RATE AND DEPTH OF PENETRATION INTO UNIFORMLY-PREPARED ASPHALT-SAND LABORATORY BRIQUETS

Test Figure No.	Treating Fluid	Penetration		
		Time, 10 Ml Vol.	Depth[1] (in.)	Time[2] into Treated Briquets
1	Control	--	--	12 sec
2	Cationic maltenes emulsion, concentrated	4 min 35 sec	0.89	18 min
3	Cationic maltenes emulsion, diluted 2:1	45 sec	0.88	16 min
4	Cationic maltenes emulsion, diluted 1:1	30 sec	0.89	18 min
5	Anionic maltenes emulsion, diluted 2:1	2 min 8 sec	1.30[3]	19 min
6	Nonionic maltenes emulsion, diluted 2:1	51 sec	0.90	22 min
7	SS-1 asphalt emulsion, diluted 2:1	--[4]	0	48 min
8	MC-70 asphalt cutback	--[5]	0.45	--[6]
9	Straight oil-resin	24 hr	0.83	8 min[7]
10	Kerosene	32 sec	0.97	2 min
11	Oil-resin in kerosene	2 min 3 sec	0.84	4 min
12	Tap water	16 hr	--[8]	21 sec

[1] Observed in ultraviolet light.
[2] For 10 ml of 0.375% Aersol OT sol.
[3] Ran through briquet and out bottom.
[4] Dried at surface forming membrane.
[5] Collapsed briquet in 11 to 12 hr without penetrating completely.
[6] Specimen damaged.
[7] Lifts surface.
[8] Not measurable.

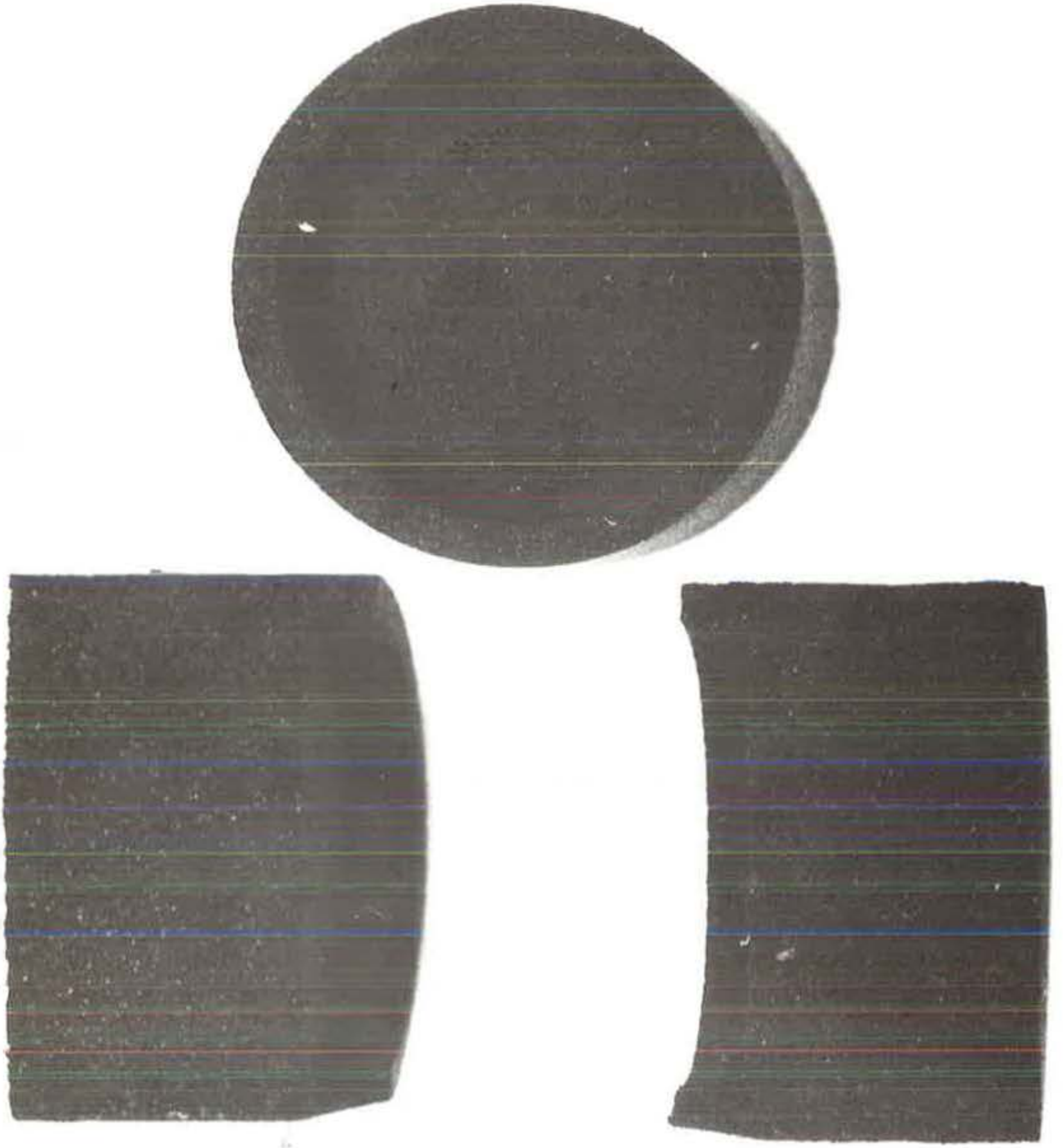

Figure 1. Untreated control.

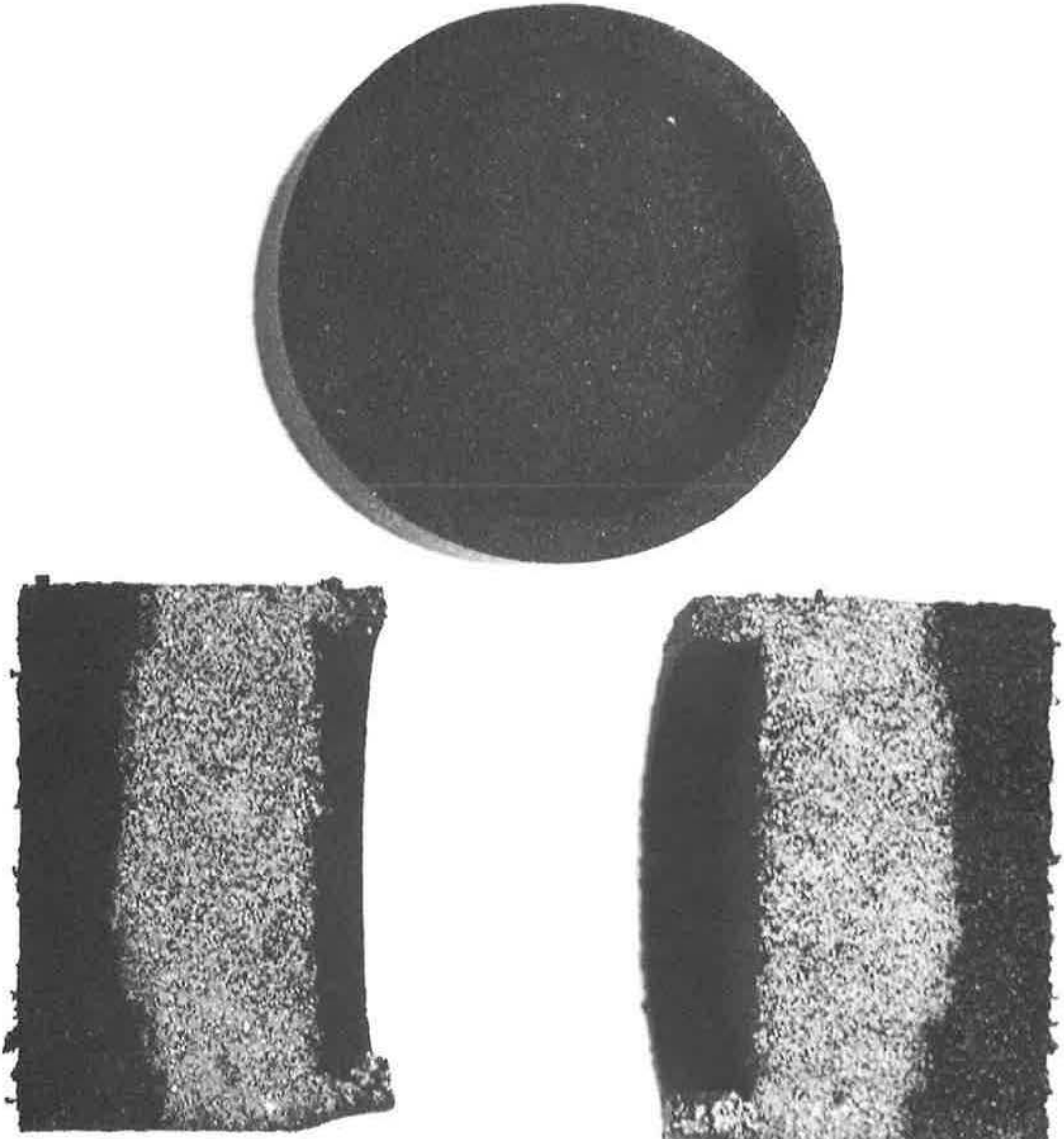

Figure 2. Cationic maltenes emulsion, concentrated.

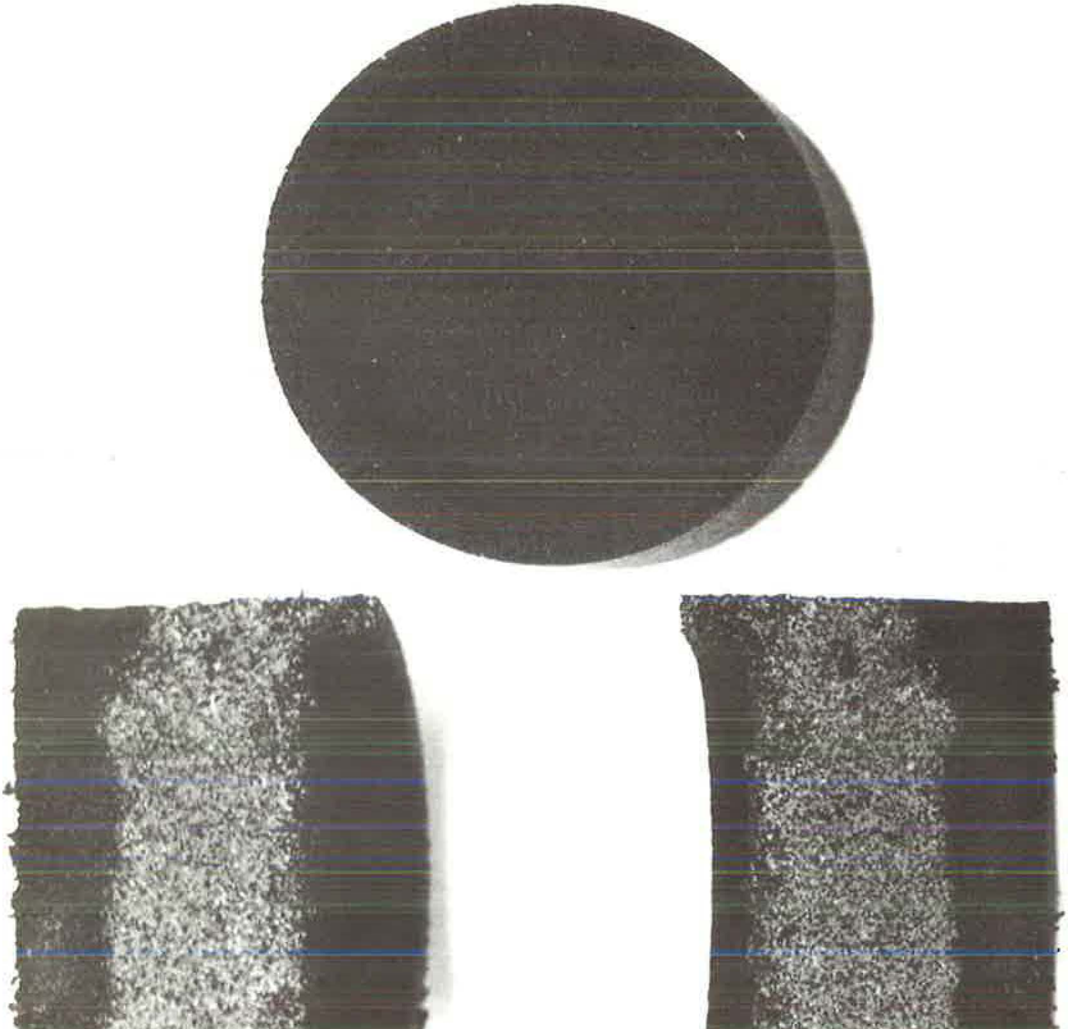

Figure 3. Cationic maltenes emulsion, diluted 2:1.

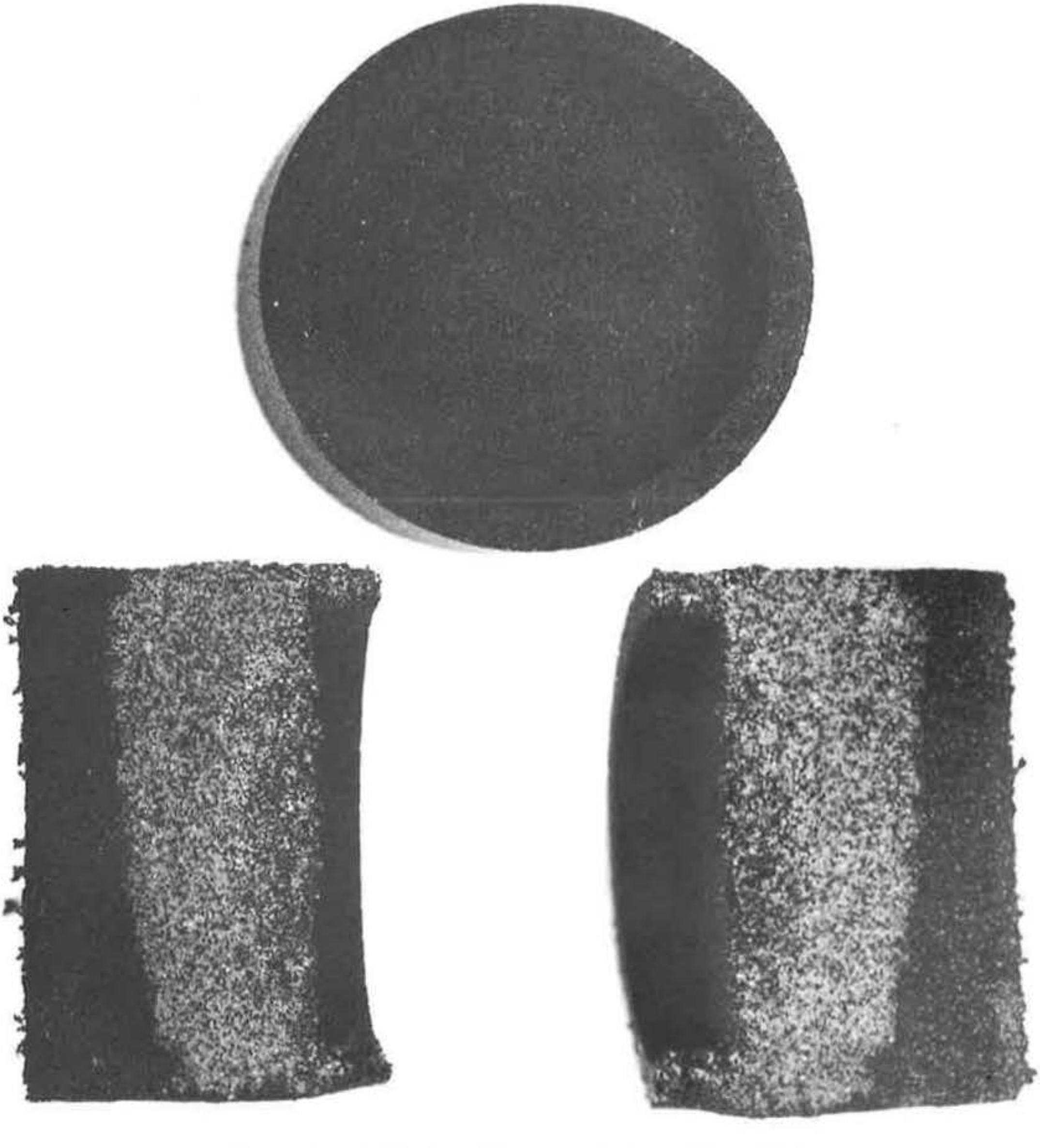

Figure 4. Cationic maltenes emulsion, diluted 1:1.

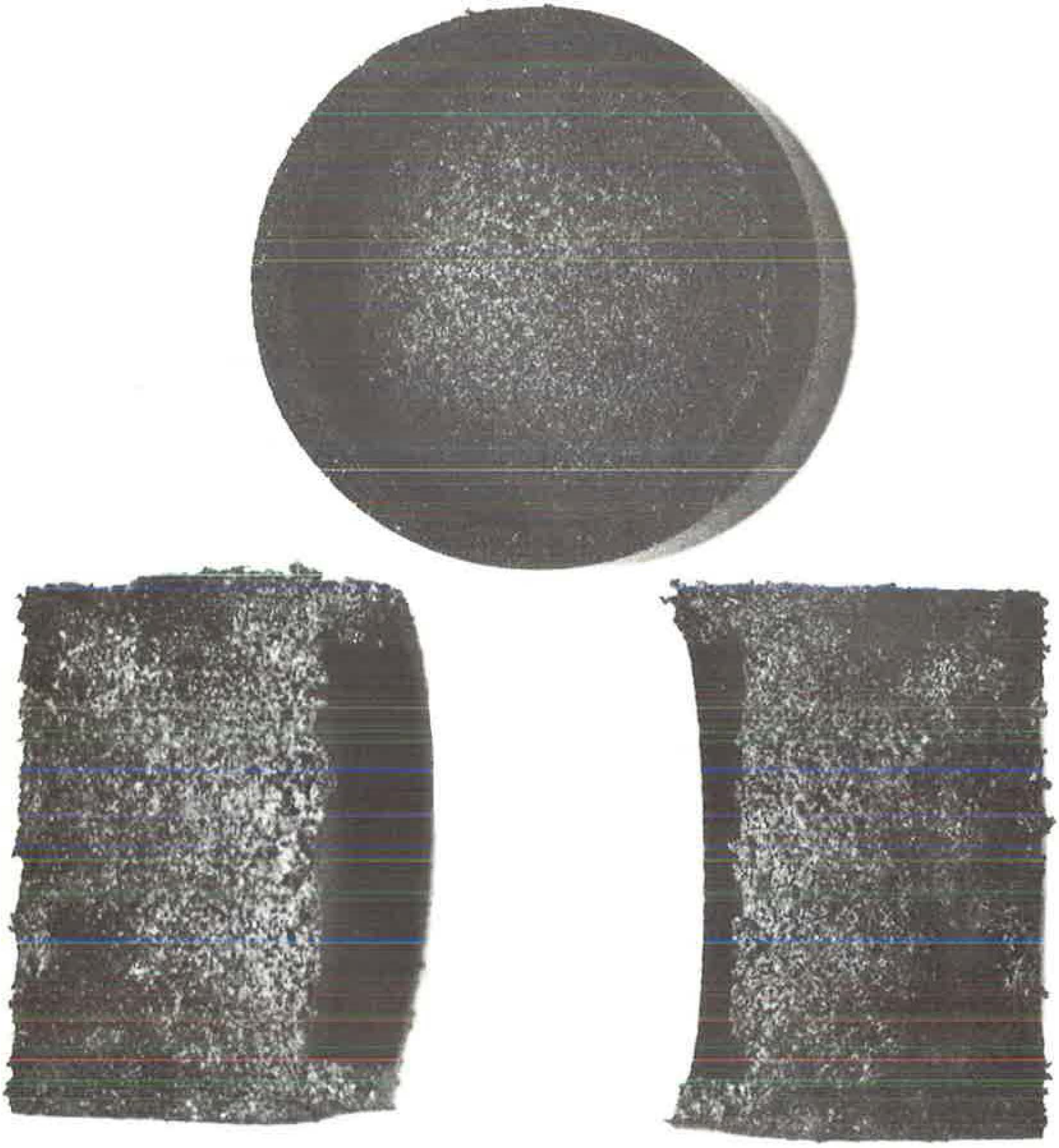

Figure 5. Anionic maltenes emulsion, diluted 2:1.

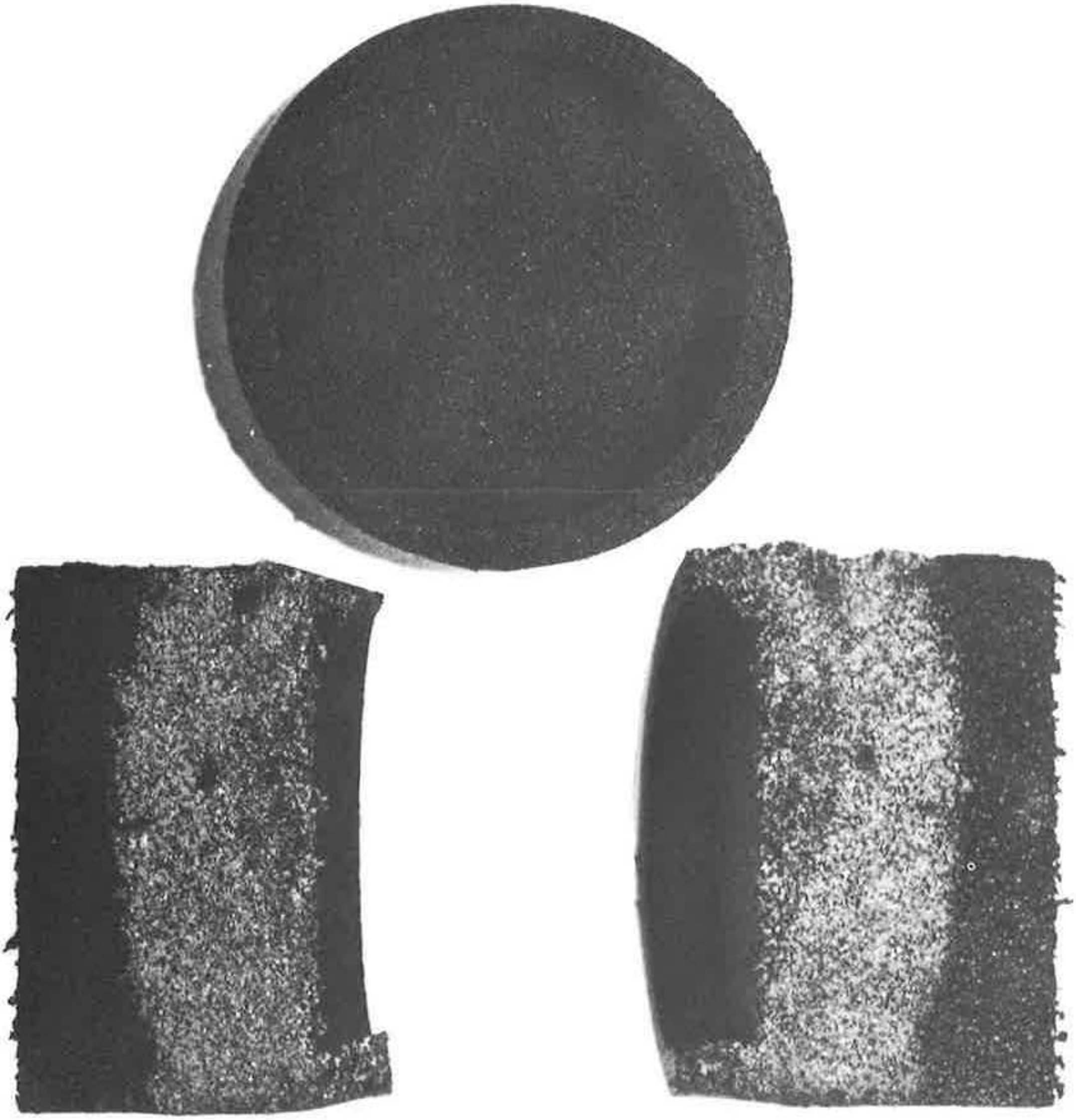

Figure 6. Nonionic maltenes emulsion, diluted 2:1.

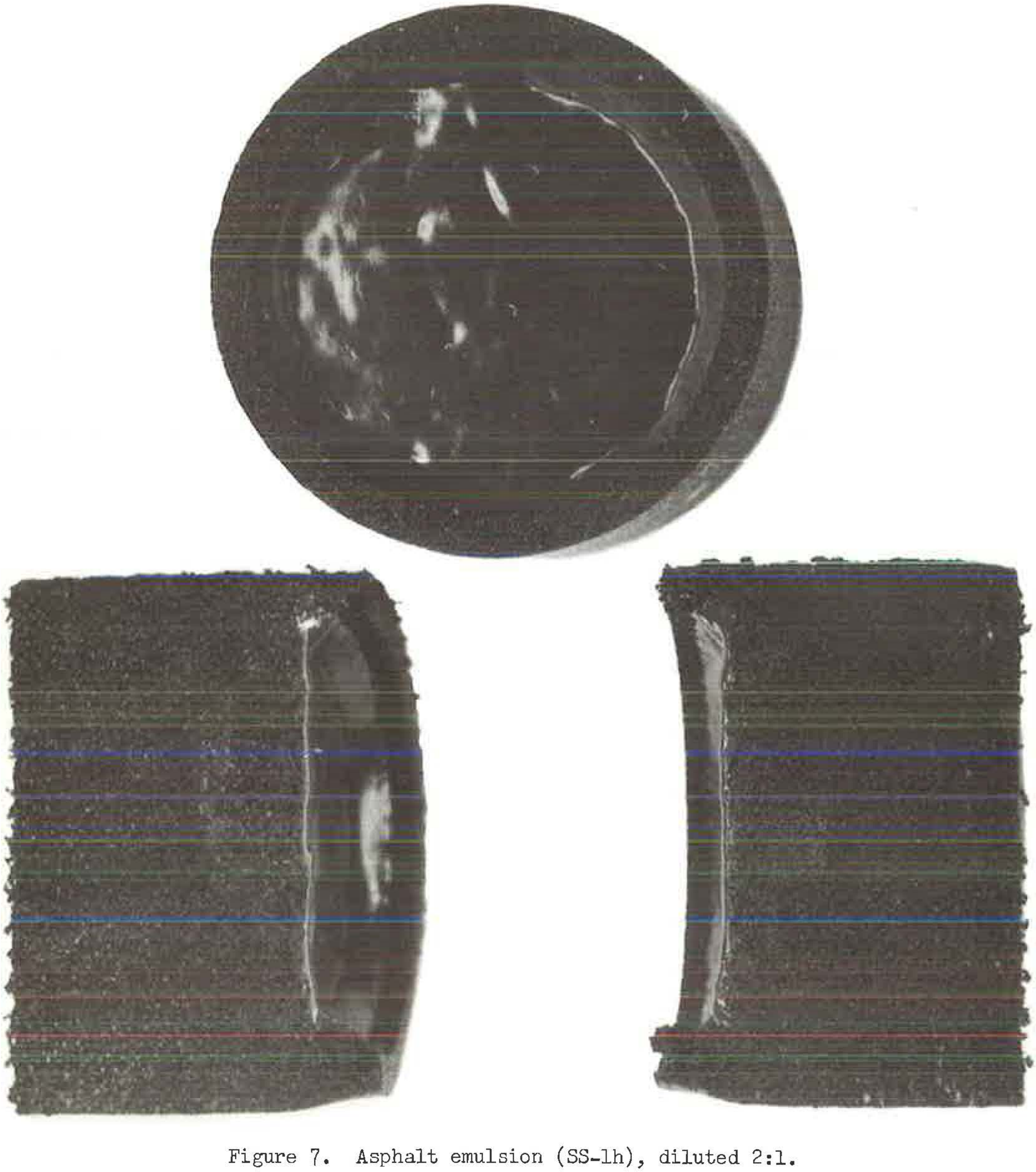

Figure 7. Asphalt emulsion (SS-1h), diluted 2:1.

Figure 8. Asphalt cutback (MC-70).

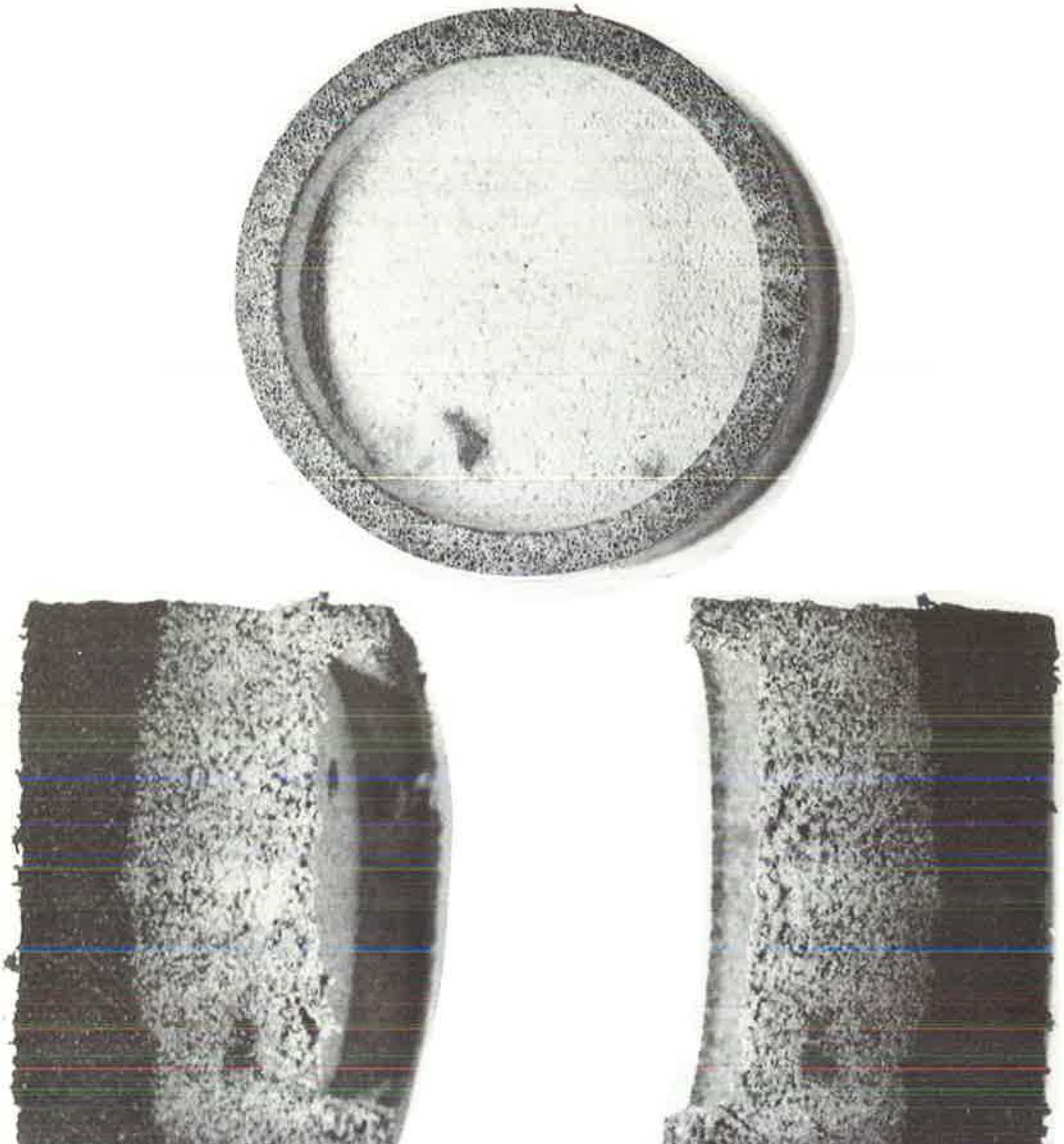

Figure 9. Maltenes without carrier.

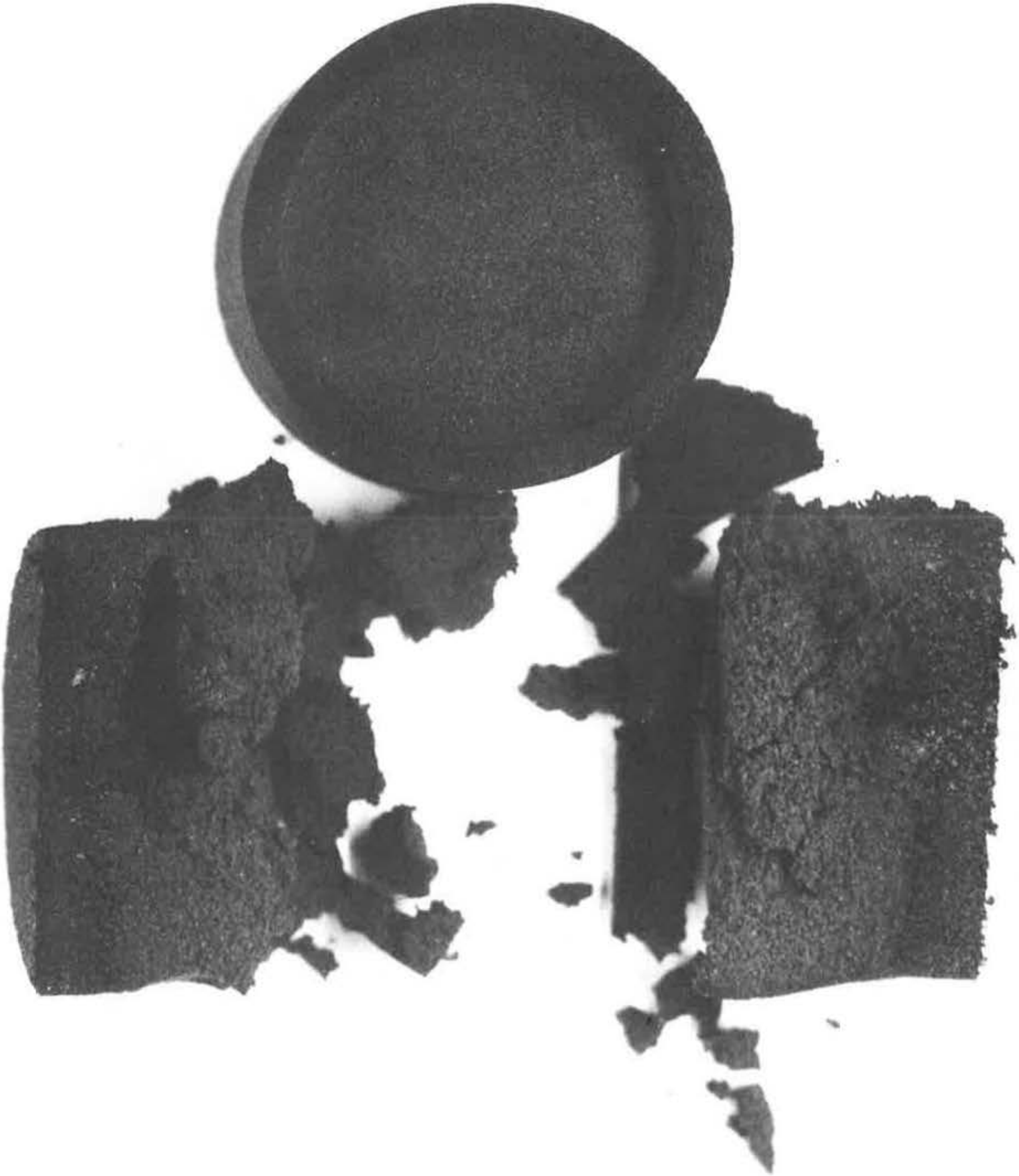

Figure 10. Kerosene.

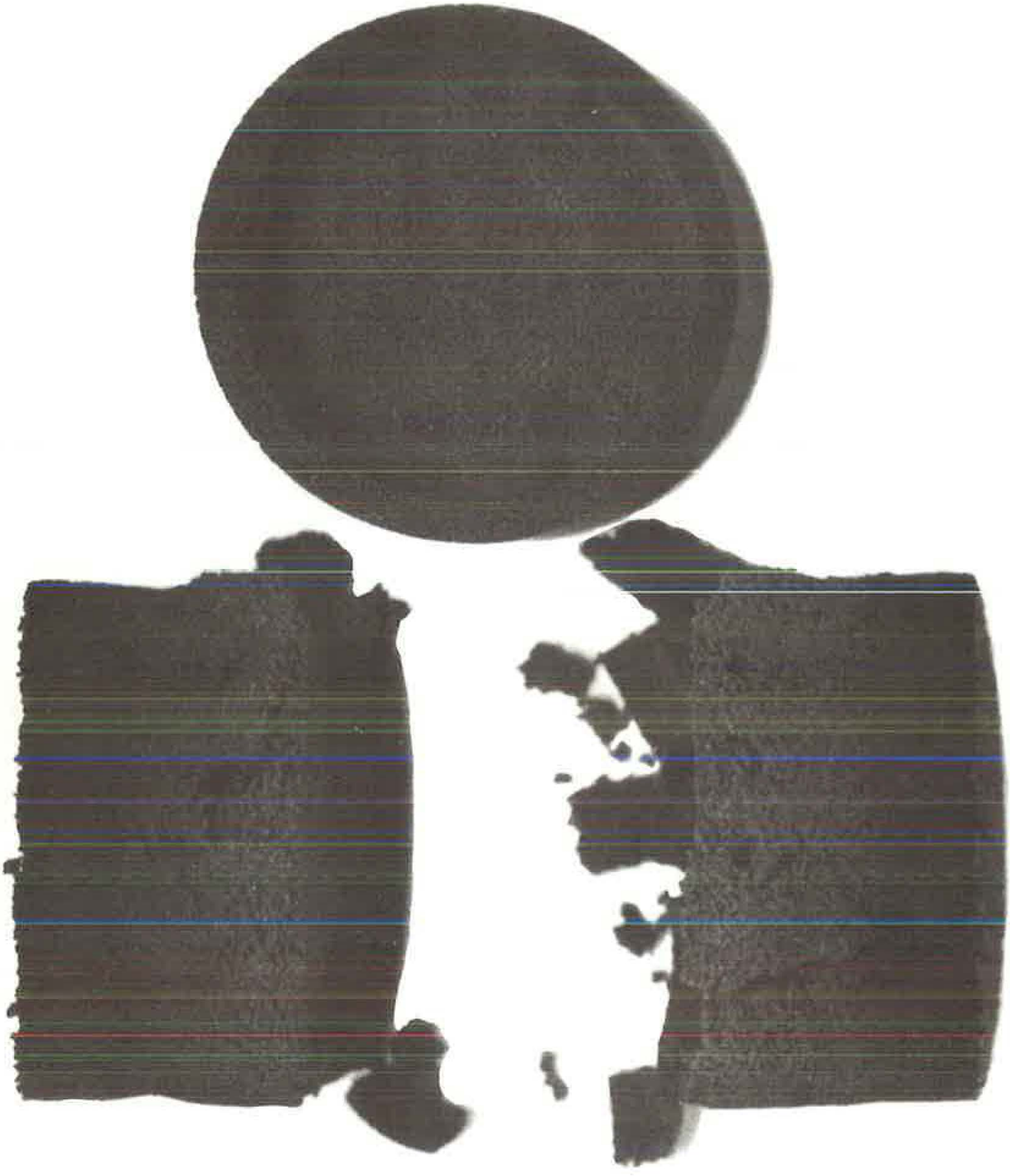

Figure 11. Maltenes solution in kerosene.

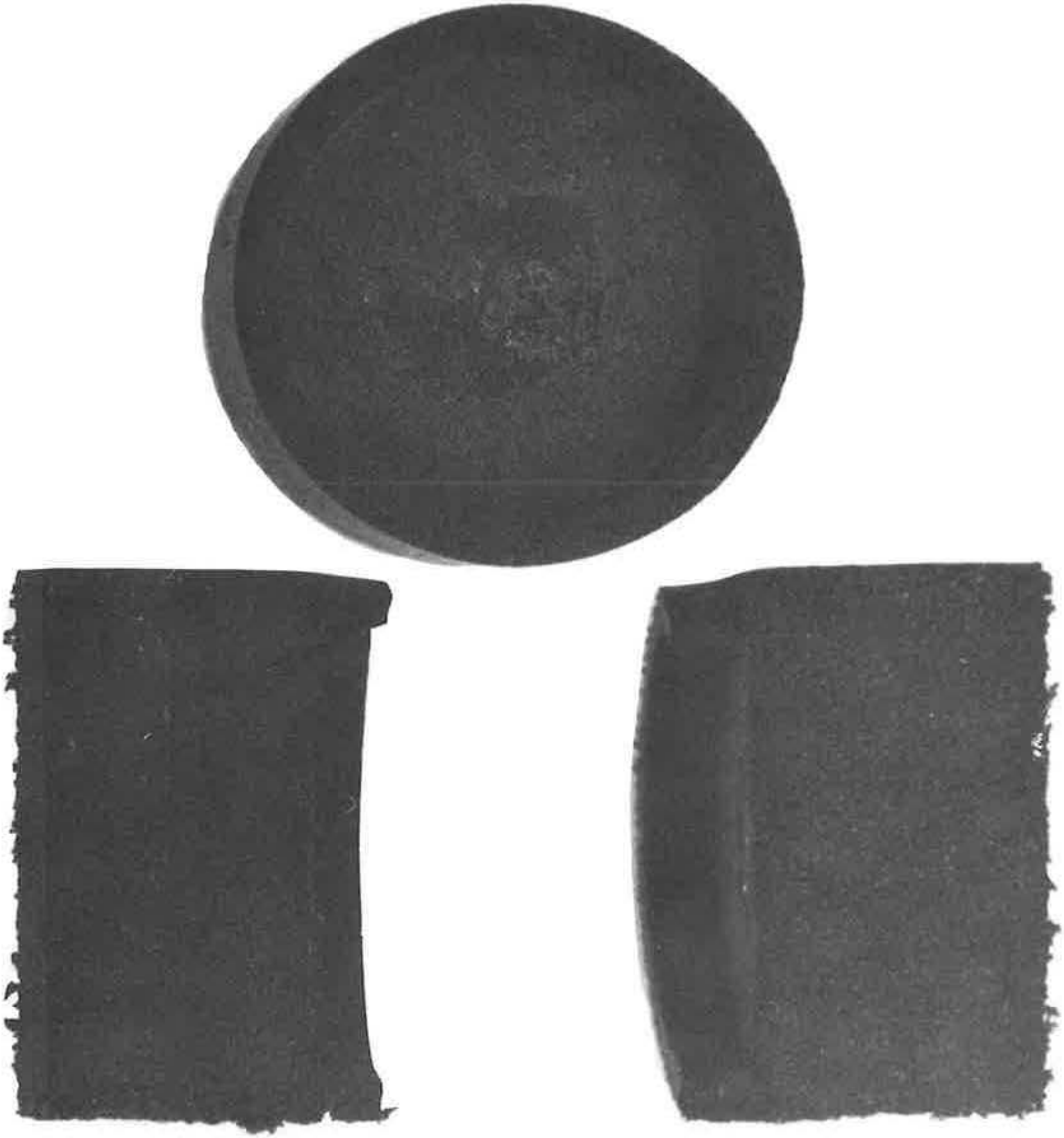

Figure 12. Tap water.

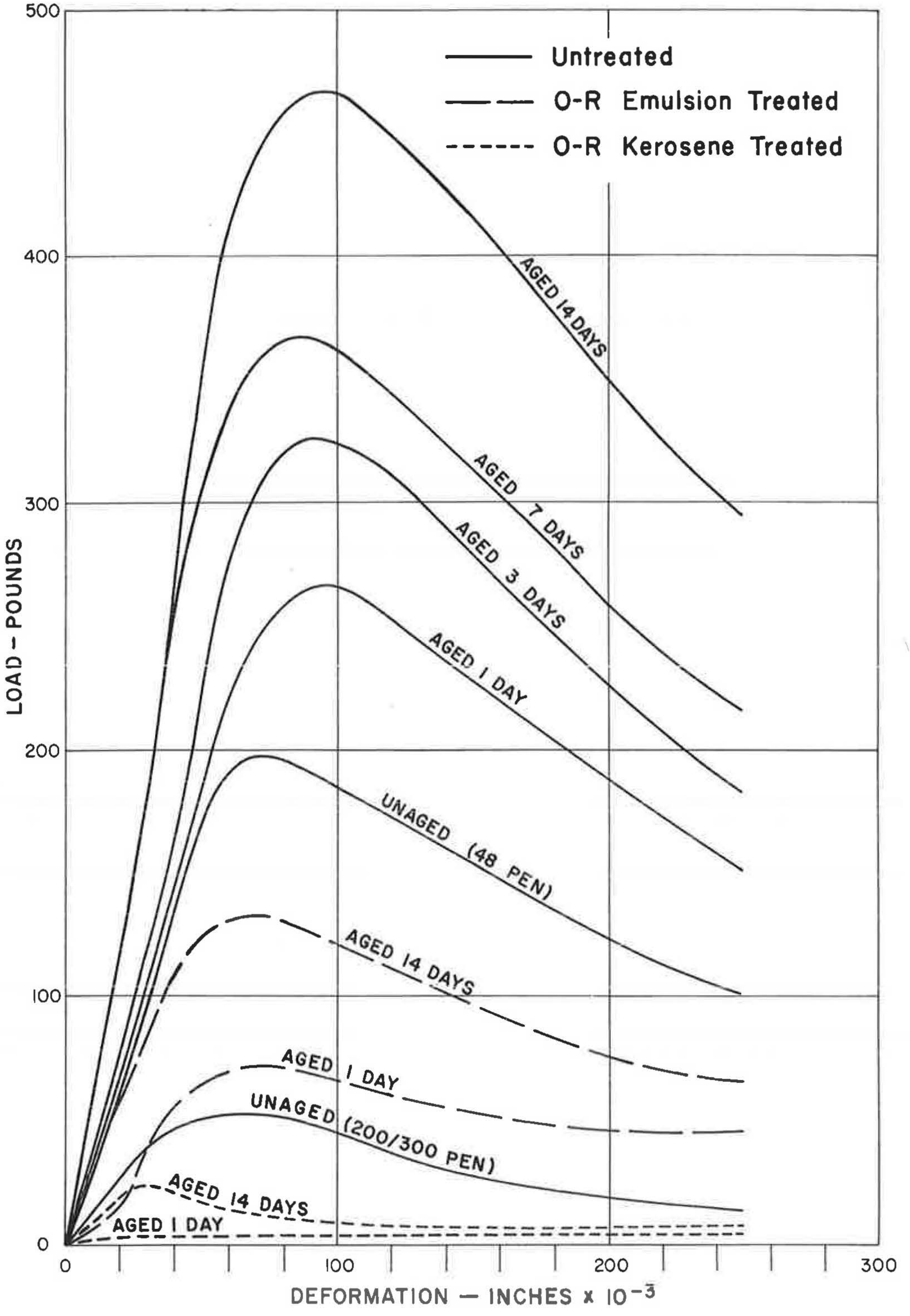

Figure 13. Effect of cationic maltenes emulsion and maltenes solution in kerosene on load-penetration test at various agings in infrared oven.

300-penetration asphalts. All other curves represent various treatments of a reference briquet made with 48-penetration asphalt at various ages. The briquets were aged in the infrared oven described elsewhere (2, App. III). The oven is automatically controlled so that the temperature within the briquets is maintained at 140 F during the specified aging time.

Figure 13 indicates that kerosene as a carrier destroys the cohesiveness of asphalt binder and that proper cohesion cannot be regained even after 14-day exposure at 140 F in the infrared oven because the kerosene has dislocated the asphalt-cement between the points of aggregate contact. In contrast, the emulsion system for introducing the maltenes does not displace the asphalt bonding the aggregate particles; in all cases, the briquets resist load penetration exceeding that of 200- to 300-penetration asphalt, although the consistency of the binder has been lowered. Figures 17 and 20 show the difference in behavior.

From similar laboratory studies and from 5 years' field experience in applying various emulsions to pavements, it has been concluded that the best method for incorporating the maltenes into the asphalt, in situ, is in the form of a fine particle, cationic emulsion as described. Although a dilution rate of two parts of emulsion concentrate to one part of water is generally used to get the best combination of penetration rate and economy, the dilution rate for a particular job should be chosen on the basis of job conditions.

LABORATORY EVALUATION OF EFFECTIVENESS

In the early stages of development, fractional chemical analysis of asphalt was used to tie in the effect of various asphalt components to performance of asphalts on the road (2, 3). This was followed by tailor-making asphalts using components from various base asphalts. It was found that certain fractions of petroleum oils and resins could not only be used to restore the original properties to aged asphalts but the same components could also make the reconstituted asphalt superior to the original asphalt in aging resistance. Figure 14 shows the improvement made in the aging resistance of a typical 200- to 300-penetration asphalt as measured by resistance to abrasion. Further laboratory tests were made to ascertain the effect of exposure of asphalt-aggregate mixes to treatment.

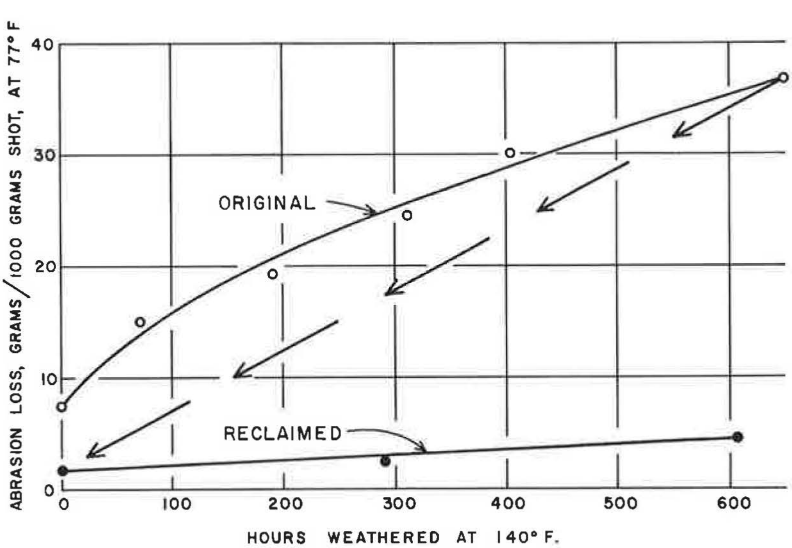

Figure 14. Durability of a 200- to 300-penetration asphalt before and after reclaiming with selected maltenes fraction (2).

Importance of Particle Charge

A series of tests was designed to determine the importance of particle charge, as related to the adhesion characteristics of the asphalt-aggregate system. The test consisted of placing 2-g pellets (made by mixing 100 parts Ottawa sand and two parts 85- to 100-penetration asphalt, and aging 7 days in the infrared oven at 140 F) into beakers containing 50 ml distilled water and heating the water to the boiling point. Several pellets were prepared and tested after treatment with 0.06 g of the oil-resin blend. The blend was applied as a cationic emulsion, as a nonionic emulsion, in an undiluted form, and as a solution in kerosene. The stripping effect was evaluated by observing the

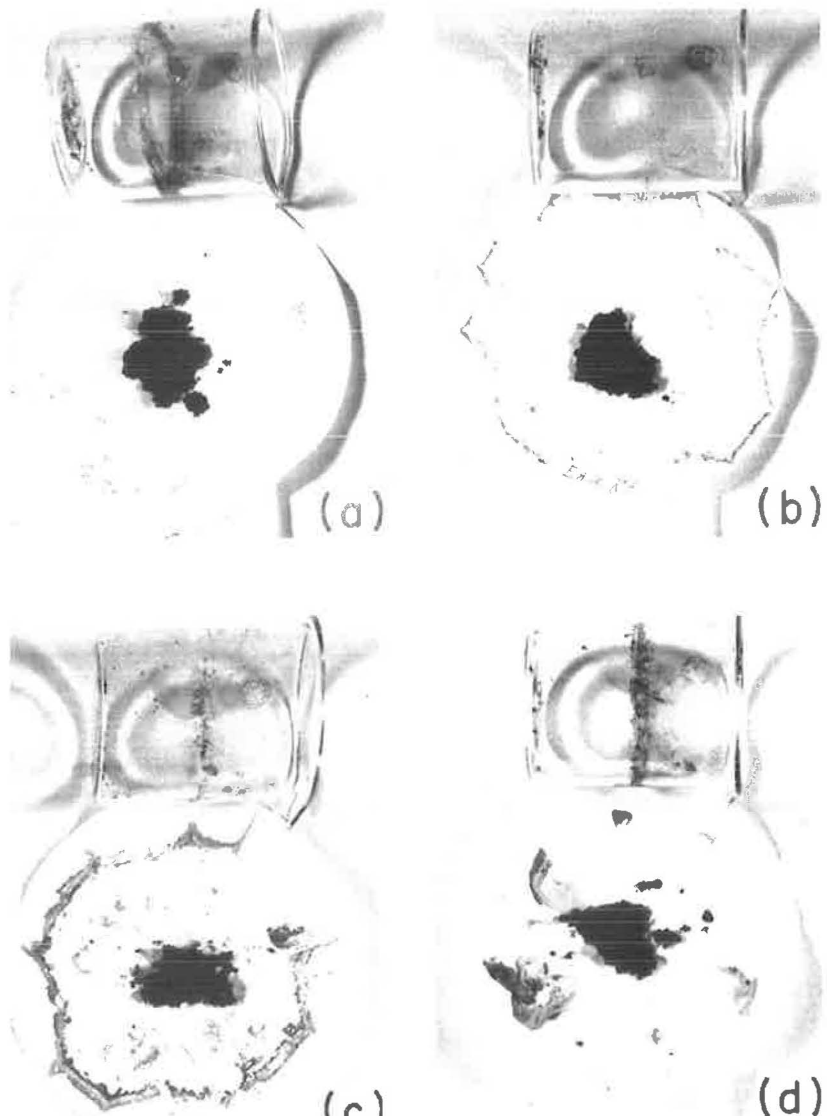

Figure 15. Effect of treatments on stripping resistance of sand-asphalt mixture when exposed to water at boiling point: (a) control, untreated; (b) cationic maltenes emulsion; (c) nonionic maltenes emulsion; and (d) maltenes solution in kerosene.

amount of asphalt floating at the water surface and clinging to the beaker walls, as well as by noting the area of exposed aggregate surfaces on the Ottawa sand at the bottom of the beaker. The contents of the beakers were then filtered through No. 1 Whatman filters.

Figure 15 shows that most of the asphalt was stripped off the sand grains in the untreated control pellet and all of the treated pellets, except the one treated with the cationic emulsion of the maltenes. Although little stripping was noted in the cationic sample, that which occurred came from asphalt at the bottom of the pellet where the treatment did not reach.

Fragments of the treated pellets were examined under a microscope to observe the effect of the treatment and aging procedure on the asphalt cementing the sand grains in the pellets. Figures 16 to 20 are photomicrographs of the specimens. The control (Fig. 16) shows the type of bond prevailing in such mixes after aging. Improvement in the bonding accomplished by the cationic emulsification system, which causes the maltenes to wet preferentially the asphalt portion of the asphalt-sand mixture, is shown in Figure 17.

Figure 18 shows that the nonionic emulsion system does little to increase the bond because there is no preferential wetting of the asphalt portion. The straight application of the oils and resins resulted in droplets of uncombined oil-resin material becoming attached to the sand grains (Fig. 19). With kerosene as a carrier for the oil-resin (Fig. 20), the asphalt films washed from the sand grains and accumulated in the voids between them, resulting in loss of bond at the points of contact.

Effect on Asphalt Durability

Another laboratory investigation was designed to ascertain the effect of the addition of the maltenes on asphalt durability and to gain some insight as to the length of time required for the oils and resins to mix completely with the asphalt. Abrasion test pellets were prepared in accordance with the procedure given elsewhere (2, App. II), using a 48-penetration asphalt and 20- to 30-mesh Ottawa sand. A set of 4 pellets was then treated with the oil-resin blend in each of the several forms under study. Two pellets were then aged in the infrared cabinet for 24 hr and the other two pellets for 7 days. Afterwards, the aged pellets were subjected to an abrasion test by tumbling them in a 16-oz French square bottle for 500 revolutions according to a procedure

Figure 16. Photomicrograph before stripping test of untreated sand-asphalt mix (see Fig. 15a).

Figure 17. Photomicrograph before stripping test of cationic maltenes emulsion treated sand-asphalt mix (see Fig. 15b).

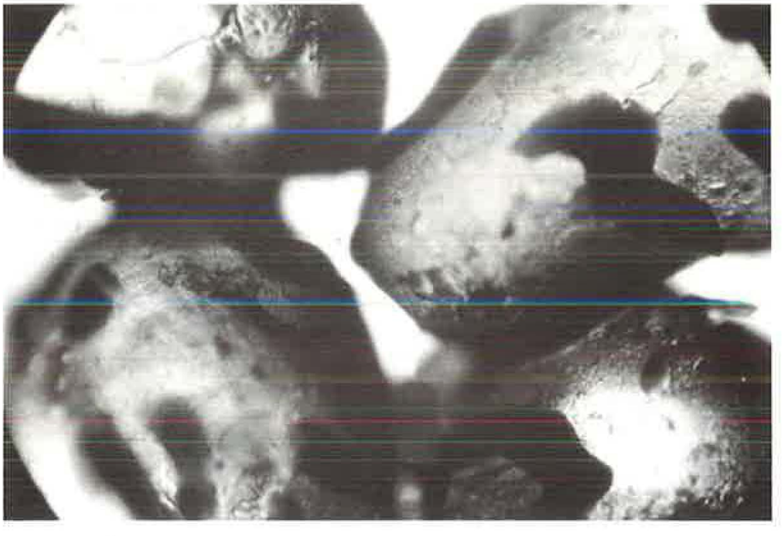

Figure 18. Photomicrograph before stripping test of nonionic maltenes emulsion treated sand-asphalt mix (see Fig. 15c).

Figure 19. Photomicrograph before stripping test of sand-asphalt mix treated directly with maltenes without a carrier.

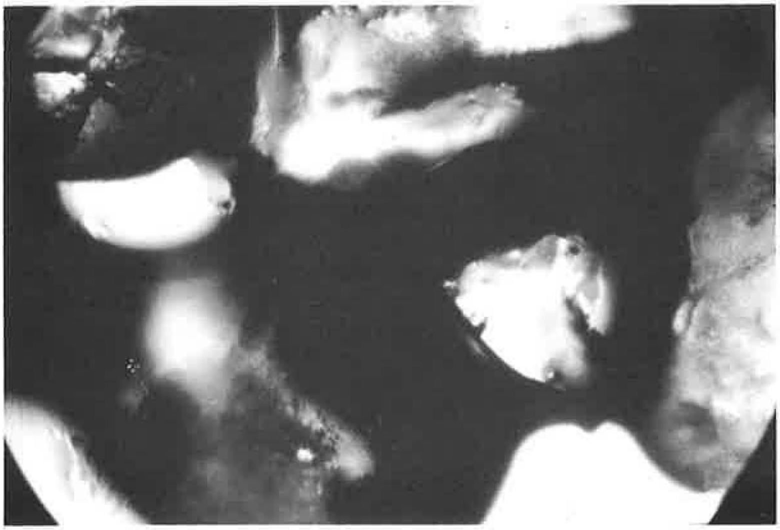

Figure 20. Photomicrograph before stripping test of sand-asphalt mix treated with maltenes solution in kerosene (see Fig. 15d).

described elsewhere (2, App. I). Table 3 summarizes the results. The selected blend of resins and oils effects an improvement in abrasion resistance irrespective of how it is carried to the asphalt. Again, the oil-resin emulsion with the cationic emulsifier shows the best long-term improvement in abrasion resistance.

TABLE 3

PELLET ABRASION TEST

Treatment	Abrasion Loss[1] (%)	
	24-Hr Aging	7-Day Aging
Untreated control	27	34
Oil-resin emulsion:		
Cationic	17	13
Nonionic	18	19
Anionic	20	21

[1] Average of duplicate specimens.

As the second part of this investigation, 8 sand-asphalt briquets, 2 ½ in. in diameter and 1.6 in. high (with the 2-in. diameter, 0.318-in. deep reservoir at the top) were molded using 48-penetration asphalt in accordance with the procedure given in Appendix A. All briquets were then weathered for 7 days in the infrared oven at 140 F.

After cooling to room temperature, one specimen was tested for load-penetration resistance as described in Appendix B. The remaining seven were then treated by pouring into the reservoir 10 ml of a dilution of two parts cationic oil-resin emulsion, and one part water. The seven specimens were again placed in the infrared oven at 140 F and withdrawn one at a time at 3-hr, 8-hr, 1-day, 2-day, 4-day, 7-day and 14-day weathering intervals. After cooling to room temperature, each briquet was tested for penetration resistance in the same manner as the untreated control specimen. Figure 21 plots the maximum resistance to penetration of the briquets vs interval of weathering after treatment.

At least 20-hr exposure in the infrared weathering oven at 140 F was needed in this case for the full effect of the cationic emulsion treatment to take place. In terms of field conditions, this would be roughly equivalent to a two-month period under average weathering conditions for the year. Accordingly, the time interval should be considerably less during the hot summer months (1 to 3 days) and somewhat longer in the

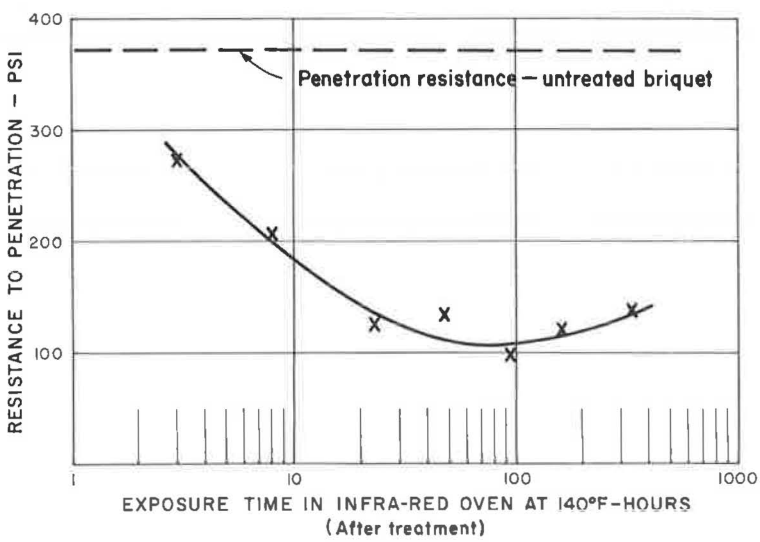

Figure 21. Load-penetration vs aging time as measure of time required for fluxing of maltenes with aged asphalt.

winter. Generally, it would be desirable to apply the emulsion when pavement temperature exceeds 80 F, even though the ambient temperature might be as low as 50 F.

OUTLINE OF USES

Although the primary function of the cationic maltene emulsion is to combine with and reconstitute aged asphalt, thereby restoring its plasticity, it acts as a "seal in depth" because after penetration it combines with and expands the asphalt. Permeability of the pavement to both air and water is reduced. In some instances, where initial shrinkage cracks had already developed, the treatment actually reversed the shrinkage process and closed the cracks. Moreover, when placed between successive layers of asphalt pavement, it provides an excellent bond by promoting a fusing of the asphalt at the interface.

Because of its ability to act as an asphalt replasticizer, a seal in depth, and a bonding agent, the oil-resin emulsion can be used on all types of asphalt paved surfaces not only in preventive and corrective maintenance but also in construction and reconstruction operations.

Preventive Maintenance

In preventive maintenance, the emulsion is applied to a structurally-sound asphalt pavement as soon as it begins to show signs of aging or brittleness through the symptoms of dryness, surface pitting, raveling or shrinkage cracking. Generally, these conditions develop in a 2- to 10-yr period after construction, depending on such factors as mix design, asphalt durability, pavement permeability and climatic conditions. The object is to penetrate the asphalt pavement and replasticize the asphalt before deterioration of the pavement has progressed too far (6).

Because asphalt pavements weather from the surface downward and it has been found that the permeability of the pavement usually increases with age, it follows that a spray treatment is, in a sense, self-regulating. The process of causing it to penetrate into the surface is, therefore, quite simple as the desired treatment consists essentially in applying that quantity of material, as determined by test, which will be absorbed by the pavement.

Exceptions to the self-regulating principle are asphalt pavements that have received surface treatments or seals which will inhibit penetration and pavements that have developed a "glaze" from high-density traffic and grease drippings. In such cases, removal of this seal is generally required to permit penetration, but this is no longer a preventive maintenance measure (7).

Corrective Maintenance

In corrective maintenance, the oil-resin emulsion is used with other procedures to improve a structurally sound asphalt pavement that is already extensively pitted or badly cracked. Usually the surface must be scraped or loosened and the loose material sprayed and recompacted or discarded. It may also be desirable to incorporate a new mixture of sand and asphalt in the loosened surface to provide a proper balance of aggregate and asphalt. If the existing surface requires a chip seal or a slurry seal, or even a thin asphalt concrete overlay, the oil-resin emulsion can be used as a spray treatment to prime the old surface and to provide a tack coat simultaneously.

The procedure to be used in corrective maintenance depends on the particular situation which determines what equipment and techniques will prove most effective. In general, combinations of heater-planing, blading, scarifying, mixing and rolling may be used, with the emulsion added at the most appropriate time.

Reconstruction

In reconstruction, the product may be used as a prime to replasticize existing asphalt-paved surfaces before a resurfacing operation or as an aid in breaking up and reworking an old, weathered asphalt paving while simultaneously replasticizing the asphalt in the mix. In the latter case, the treatment may be sufficiently effective to make the mix reusable as a surfacing.

New Construction

In new construction, the object of the treatment is to insure plasticity and improve the durability characteristics of the freshly-placed mix, while providing a construction seal to reduce permeability. This treatment applied after the asphalt paving has been spread and compacted serves two purposes: (a) it penetrates the surface and combines with the asphalt to restore the durability lost in the mixing cycle at the hot plant; and (b) it combines with the asphalt and causes the asphalt to expand and block the pores of the pavement, thereby sealing the pavement to the depth of penetration (Fig. 22).

A light spray of the oil-resin emulsion between lifts of asphalt during construction also serves as an excellent tack coat. By fusing with the asphalt on both sides of the interface, a positive joining of asphalt layers is developed.

APPLICATION QUANTITIES AND METHODS

The simplest method of bringing together the oil-resin blend and the aged asphalt is by spraying a predetermined quantity on the pavement surface and allowing it to soak in. This procedure is always effective in preventive maintenance, in new construction seals, and in priming and tack coating operations. Conventional asphalt spreader trucks may be used but they must be free of leaks and well calibrated to give a measured and uniform spread.

In the case of preventive maintenance and prime coating, the rate of spread is determined by the "grease ring" permeability test procedure (Fig. 23) described in Appendix C, using the criterion that the minimum spread rate is that quantity which soaks into the pavement in a 15-min period. In new construction seals, a spread rate of 0.2 is generally used for a 2-in. lift or more, and about 0.05 gal per sq yd is used in tack coating.

Figure 22. Asphalt section treated with cationic maltenes emulsion as construction seal: after rainstorm treated section dried quickly (background); untreated portion retains water in pores (foreground).

Figure 23. "Grease ring" permeability test.

Figure 24. Spraying cationic maltenes emulsion in preventive maintenance; right lane lightly sanded.

It has been found that all newly-laid asphalt paving and the majority of weathered asphalt pavements are sufficiently permeable to respond to this simple spray-and-penetrate approach, provided a seal coat has not been applied.

If the pavement has had a so-called "fog" seal of asphalt emulsion or RC cutback, there is usually no difficulty in getting penetration, as this type of seal weathers rapidly and wears off in a short time. However, the fog seal does present a problem calling for special precautions as its weathered remnants will combine with and hold a certain amount of oil-resin residue at the surface — causing a slippery condition. In such instances, a light sanding is necessary (about 1 to 2 lb of fine, dry sand per sq yd) before traffic can be allowed on the pavement. In no case, however, should this sanding be done before the emulsion has had at least 15 min (preferably 45 min) to penetrate.

The sanding procedure is also appropriate when it is difficult to control traffic speed or when traffic must use the treated surface before complete penetration. It should be used on all areas where grease drippings have accumulated, such as at intersections, steep up-grades or sharp turns. Where light sanding is required over the entire area and when time is of the essence, sanding should begin from 15 to 45 min after the oil-resin emulsion is applied and follow the path of the application at the same speed (Fig. 24). The sand must be dry and gritty.

Where chip seals and slurry seals have been applied, penetration is often not possible in a reasonable time period unless the seals have largely worn away or are burned or otherwise removed by a heater-planer or similar operation. However, sometimes it is desirable to treat a chip seal, itself, in order to rejuvenate its asphalt. This is not only possible but readily accomplished, provided the rates of spread are predicated on existing conditions. Slurry seals as a rule do not respond well to treatment, particularly if they are extremely dense. Moreover, slurry seals are often overly rich in asphalt and present a skidding hazard when treated.

Where the maltenes emulsion is used in reconstruction as an aid in breaking up an existing pavement so that it can be reworked, respread and recompacted, a successful procedure (Tulare County, Calif., 1961-62) was to shoot the road mix, one lane at a time, with 0.1 gal per sq yd of a 2:1 dilution of the emulsion and water a few days before discing. The oil-resin blend replasticized the hard ¾-in. thick crust on the old road mix enough to speed up the discing and to facilitate break-up of the fragments of crust into a workable mixture, free of dry balls of material, by simple blade mixing. Over 100 lane-miles of road mix were reconstructed in this fashion with considerable savings in time, labor and equipment. The work crew was able to cover about twice the area in a given period of time and the discs required sharpening less frequently. The finished pavement was superior in serviceability, appearance, and smoothness as there were no hard lumps of unbroken material protruding from the surface.

EVALUATION OF FIELD TEST SECTIONS

Many field test sections are under observation to ascertain both the short- and long-range effects of various methods of treatments using this particular emulsion. Provisions were made for comparisons of treated and untreated areas subjected to the same loading and weathering conditions.

In examining and evaluating these test areas, the important consideration is to judge the relative overall performance of the treated vs untreated pavement, rather than to rely solely on visual appearance. Evidence of changes in surface texture, or cracking, or other signs of embrittlement should be sought and verified by digging into the pavement to compare effects beneath the surface. Actual field measurements of relative pavement permeability and laboratory test data on asphalts recovered from cores taken from field test sections can provide the relative numbers for a true assessment.

The most notable example of preventive maintenance is a test section on a taxiway at Edwards Air Force Base, Calif. In June 1959, the asphalt concrete paving was beginning to show signs of surface pitting but no shrinkage cracks had developed. Spread rates of 0.15 and 0.30 gal per sq yd of a four part emulsion to one part water dilution were applied on both ends of an untreated control section.

In the last $3\frac{1}{2}$ years, the surface of the untreated control section has pitted severely, the asphalt has been stripped from the aggregate at the surface and large shrinkage cracks have gradually developed and expanded. A recent survey found that the sides of the shrinkage cracks are breaking off and spalling badly because of extreme brittleness. These cracks meander over to the boundary of the treated area and gradually fade out (Fig. 25). To date, no cracking has started in the treated test areas.

Preventive maintenance test areas are also located on the asphalt concrete apron at Meadows Field (the Bakersfield airport), on the asphalt concrete resurfacing of the White Wolf Grade section of the California State Highway System east of Arvin (Fig. 26), on some asphalt concrete paved shoulders of highway US 99 just north of Lodi, on a road mix on Kern County's Comanche Road just north of US 466 (Fig. 27), and many other locations representing various types of asphalt surfaces.

There are also several test sections employing the cationic maltenes emulsion as a construction seal, including Kern County's Panama Lane project.

Lodi Test Section

In August 1961, the 8-ft wide asphalt concrete outer shoulder of US 99 north of Lodi was treated with the emulsion. The emulsion concentrate, as delivered to the jobsite, was diluted 2:1 with water and applied with a conventional asphalt distributor. Rates of spread of 0.06 and 0.14 gal per sq yd and an excess quantity (approximately 0.25) were used for this test. For comparison, a section of the shoulder was sprayed with 0.1 gal per sq yd of a conventional asphalt emulsion (SS-1), diluted in the proportion of 60 percent emulsion and 40 percent water.

It was apparent that the diluted oil-resin emulsion penetrated into the five-year-old pavement quite readily — complete penetration within 5 to 20 min depending on the rate of spread. Because the pavement had not previously been sealed and there were no

Figure 25. Taxiway at Edwards AFB viewed from treated area toward untreated control section.

Figure 26. White Wolf Grade section six months after treatment: treated lanes in background; untreated lanes in foreground.

Figure 27. Weathering of road mix more rapid than for asphalt concrete accounting for contrast in surface texture nine months after application: treated lane, right; untreated lane, left.

TABLE 4

TEST RESULTS ON RECOVERED ASPHALTS FROM CORES, LODI SHOULDERS PROJECT, US 99
(Age: 2 Months)

Depth of Slice (in.)	Viscosity of Recovered Asphalt, Megapoises				
	Untreated	Cationic Maltenes Emulsion[1]			Asphalt Emulsion[1] SS-1h, 0.1 Gsy
		0.06 Gsy	0.14 Gsy	0.25 Gsy[2]	
0 - 3/8	86 (12)	7 (36)	0.35 (143)	0.2 (185)	45 (15)
1/2 - 7/8	21 (21)	19 (22)	11.4 (29)	6.3 (38)	16 (24)
1 - 1 3/8	19 (22)	15 (25)	10.5 (31)	8 (34)	15 (25)
1 1/2 - 1 7/8	--	--	--	11 (29)	--

Note: numbers in parentheses are penetration values of asphalts obtained from conversion chart.
[1]Diluted 2:1.
[2]Approximate.

accumulations of grease drippings, no sanding was required. On the other hand, the asphalt emulsion did not penetrate and sanding was required for safety and to prevent pick-up of the asphalt by traffic.

Two months after application, cores were taken from the various sections and cut horizontally at 1/2-in. intervals. Viscosities of the asphalt recovered from each slice were then determined using the sliding plate microviscometer (Table 4). Figure 28 shows the asphalt penetration values as the ordinate instead of viscosity as penetration value is a more familiar term to highway engineers.

A significant change in asphalt viscosity took place in the pavement to a depth of at least 1/2 in., with a possible effect of the added maltenes to a depth of 1 1/4 in. for the heavier treatments (Fig. 28). Inasmuch as these results were obtained relatively soon after application, it is possible that the oil-resin blend had not had sufficient time to reach a state of equilibrium in depth with the asphalt.

It is also apparent (Table 4 and Fig. 28) that the asphalt emulsion treatment has not influenced the aged asphalt within the pavement in any way. Even the viscosity of the asphalt recovered from the top slice of the core, which includes the asphalt seal, was not significantly greater than the viscosity prior to treatment. This indicates that an asphalt emulsion fog seal should not be considered as a pavement rejuvenator but as a surface dressing.

Meadows Field Test Section

In July 1960, a cationic maltenes emulsion test section was placed on the asphalt concrete light-plane parking area at Meadows Field (the Kern County Airport). Two 230-ft long, 8-ft wide parallel strips were treated with a 1:1 dilution at rates of spread of 0.11 and 0.22 gal per sq yd. Because this was one of the first test sections placed on a commercially-used area, the treatment was deliberately kept on the light side as a precautionary measure. However, penetration was rapid and complete; within 10 min all of the diluted emulsion had penetrated and the area was ready for traffic.

The water permeability test, Calif. Test Method 341-A, (8) was run 24 hr later and results averaged 20 ml per min on the untreated area and 10 ml per min on both treated sections. A value of 10 ml per min indicates that the pavement is impervious to water.

The original four-year-old pavement was dry in appearance at the surface and beginning to ravel slightly. Shrinkage cracks were beginning to form in the characteristic progressive, right-angled pattern. In October, it was observed that the shrinkage cracks had closed completely in the two treated strips but still remained in the surrounding untreated areas. Figure 29 shows the test area in October 1960 with the two parallel strips with all shrinkage cracks in the vicinity outlined with white chalk. The shrinkage cracks in the narrow strip between the two test sections did not close. Figure 30 shows one of the characteristic shrinkage cracks between the two test strips.

As an immediate result of this observation, a full-scale treatment of the entire light-plane parking apron with the emulsion was carried out in October 1960, except for a 30- by 50-ft section retained undisturbed for a long-term study. A spread of 0.20 gal per sq yd of a 2:1 dilution was used, a somewhat heavier treatment than the heaviest treatment of the two test strips, inasmuch as the lower dilution with water provides more oils and resins at the same rate of spread. This increase was made because other projects had indicated that the original applications were on the conservative side.

In July 1962, two 6-in. diameter cores were taken from each of the two test strips and from the adjacent untreated area. Table 5 gives the results of various laboratory tests. The maltenes emulsion treatment was most effective in

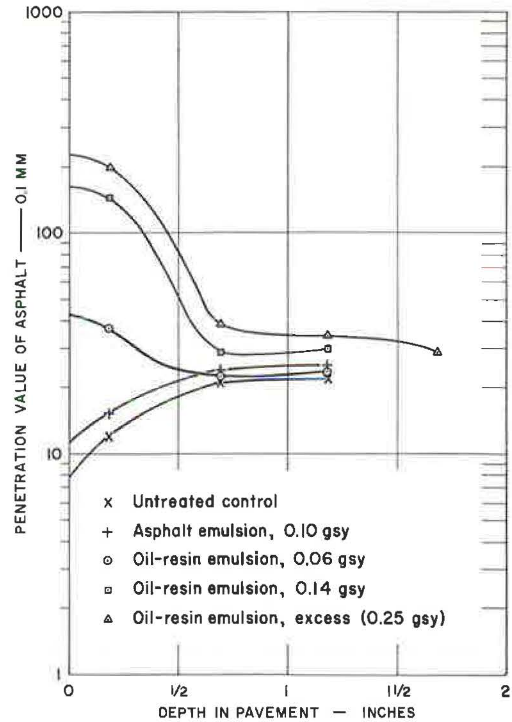

Figure 28. Change in asphalt penetration value with depth in pavement, Lodi Shoulder Project.

Figure 29. Meadows Field Test Section four months after treatment: absence of shrinkage cracking in treated areas; chalk outlines cracking still existing in surrounding untreated area and in untreated area between passes.

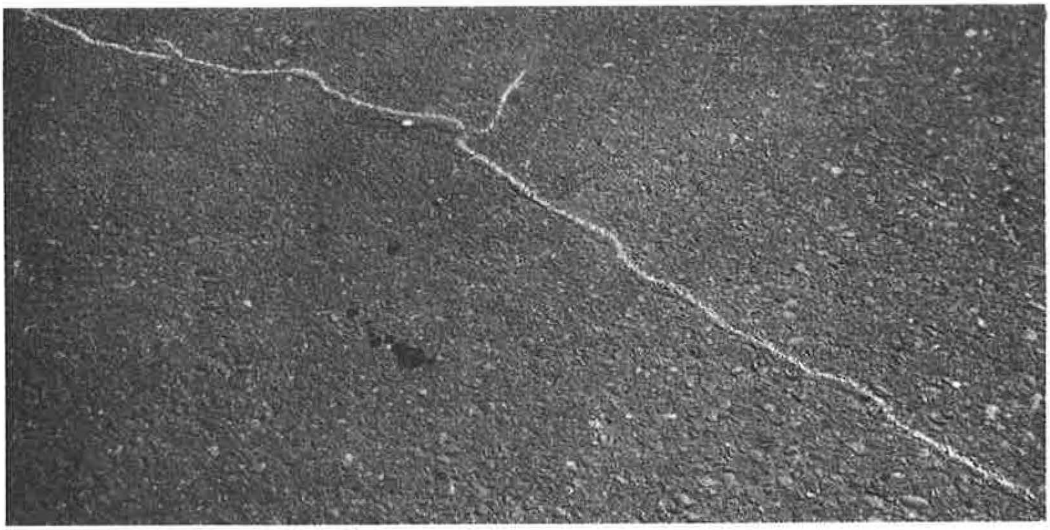

Figure 30. Shrinkage crack between the two test strips (Fig. 29).

TABLE 5

TEST RESULTS ON RECOVERED ASPHALT FROM MEADOWS FIELD CORES (Age: 2 Years)

Determination	MF-Untreated		MF-Oil-Resin Emulsion Treated			
			0.11 Gsy		0.22 Gsy	
	Top	Middle	Top	Middle	Top	Middle
Asphalt content[1] (% agg. wt.)	3.8	4.4	5.2	4.9	5.0	5.0
Viscosity (megapoises)	37.3	14.3	14.0	24.3	2.7	22.0
Penetration value (0.1 mm)	17	26	26	20	56	22
Chemical composition[2] (% by wt.):						
Asphaltenes	25.8	20.6	25.5	21.9	24.9	22.8
Nitrogen bases	31.9	35.3	29.7	34.6	28.1	34.7
First acidaffins	10.2	12.8	10.9	12.7	10.4	11.9
Second acidaffins	18.3	17.2	18.7	16.7	19.9	16.2
Paraffins	13.8	14.1	15.2	14.1	16.7	14.4
Pellet abrasion test (% loss in wt.):						
As recovered	64	27	8	42	1	42
Aged	100	72	39	88	4	90
Maltenes distribution ratio[3]	1.31	1.54	1.20	1.54	1.05	1.52

[1] By Soxhlet extraction.
[2] Rostler method.
[3] $(N + A_1)/(P + A_2)$.

increasing and holding the asphalt content in the top portion of the pavement at a value of about 5 percent, while the asphalt content in the surface of the untreated pavement has dropped 3.8 percent indicating that disintegration can be expected to occur rapidly.

After a 2-yr period in the severe weathering conditions of the southern San Joaquin Valley, the treatment still shows a beneficial effect as evidenced by the lower viscosity and higher penetration values of the asphalt from the top sections of the treated core compared to the untreated cores. Even the relatively light treatment of 0.11 gal per sq yd of 1:1 dilution has improved the asphalt in the top portion of the treated pavement over that in the middle section.

It appears, however, that neither of the test sections had enough material applied to penetrate into and in any way influence the properties of the asphalt in the middle slice. This is substantiated by the essentially constant chemical composition of the three middle sections, particularly the paraffins content, and the maltenes distribution ratio. Definite effect of treatment, however, is shown by the change in chemical composition of the top sections of the three cores where a general increase in the percentages of second acidaffins and paraffins is apparent, and the maltenes distribution ratio is successively decreased from 1.31 to 1.20 to 1.05 by the treatments with 0.11 and 0.22 gal per sq yd, respectively.

The significance of the maltenes distribution factor as a measure of asphalt durability is shown in Figure 31 (3) which is based on abrasion test data on over 100 samples of 85- to 100-penetration grade asphalts vs their maltenes distribution factor, $(N + A_1)/(P + A_2)$. Rostler and White (3) have shown that a maltenes distribution factor of less than 1.14 characterizes an asphalt of excellent durability as measured by abrasion resistance.

Recovered asphalts from the Meadows Field cores were also subjected to the pellet abrasion test to ascertain what effect the treatment had on asphalt durability. These values, also given in Table 5, indicate the improvement of the treated asphalt in resisting further aging.

Panama Lane Test Project

On May 1, 1961, a cationic maltenes emulsion treatment was applied on 600 ft of freshly-spread asphalt concrete placed between Stations 118+00 and 124+00 on the westbound lane of Panama Lane in Kern County. The asphalt concrete specified on

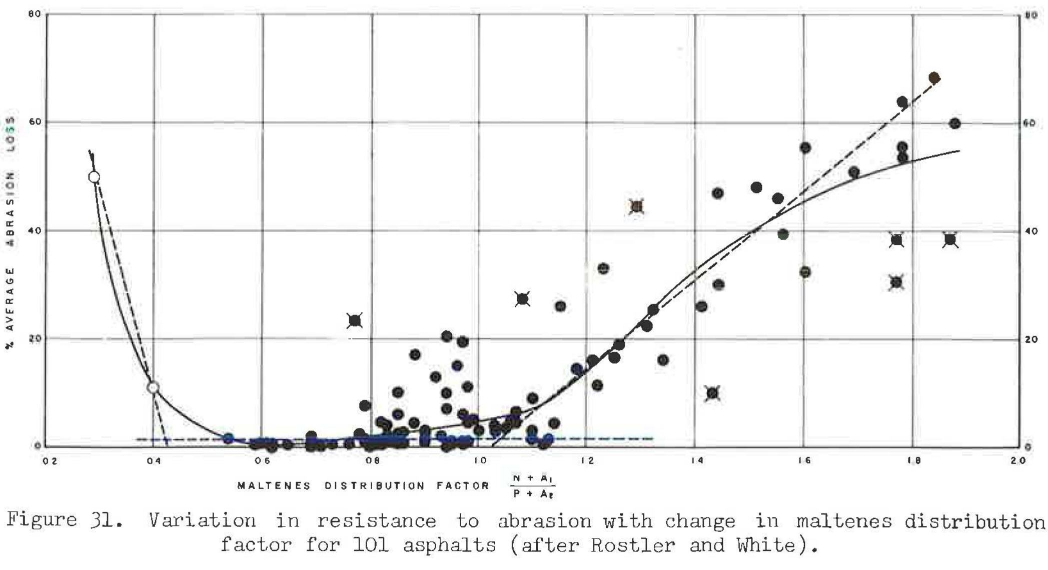

Figure 31. Variation in resistance to abrasion with change in maltenes distribution factor for 101 asphalts (after Rostler and White).

TABLE 6

PHYSICAL AND MECHANICAL PROPERTIES OF ASPHALT MIXTURE, PANAMA LANE PROJECT

Determination	Surface Course							
	Untreated				Oil-Resin Emulsion Treated (0.1 gsy)			
	Sta. 119	Sta. 120	Sta. 122	Avg.	Sta. 119	Sta. 120	Sta. 122	Avg.
Bulk specific gravity[1]	2.29	2.28	2.27	2.28	2.29	2.29	2.29	2.29
Hveem stabilometer[1] value	33	33	34	33	33	30	30	31
Cohesiometer[1] value	155	115	144	138	134	123	126	128
Extracted asphalt content (% wt. agg.)	5.6	5.5	4.7	5.3	5.9	6.0	5.5	5.8
Aggregate gradation (% passing sieve):								
3/4	100	100	100	100	100	100	100	100
3/8	71	70	58	66	73	74	65	71
No. 4	56	57	45	53	58	62	54	58
No. 8	45	47	37	43	48	50	44	47
No. 30	21	21	8	17	21	22	20	21
No. 200	4	3	4	4	4	3	3	3

[1]On laboratory molded specimens.

this project consisted of a 3/4-in. medium grading type A aggregate combined with 5.0 percent 85- to 100-penetration grade asphalt, with all materials and procedures meeting the requirements of Section 39 of California Standard Specifications. Total thickness of asphalt concrete was 3 in., placed in two 1 1/2-in. lifts.

The first 1 1/2-in. lift was spread and compacted in the usual manner, except that three complete coverages with a 25-ton pneumatic-tired roller were used in lieu of the rubber-tired rolling called for in Section 39, as this was also an experimental rolling project. While the mechanical spreader was placing the second 1 1/2-in. lift, a distributor truck applied 0.1 gal per sq yd of a 4:1 dilution of the emulsion to the first lift about 200 to 800 ft ahead of the mechanical spreader. Penetration into the first lift was complete in 2 to 5 min.

The second lift was placed over the treated first lift. While the mix was still hot (255 F), an additional 0.1 gal per sq yd of a 4:1 dilution was then applied by offset spreading after laydown and before the breakdown pass of a 12-ton steel wheel roller. The material soaked into the semi-compacted mix in less than 30 sec. Following breakdown, three passes were made with the 25-ton pneumatic roller with final rolling accomplished with a 10-ton steel-tired tandem roller.

TABLE 7

TEST RESULTS ON RECOVERED ASPHALTS FROM PANAMA LANE TEST PROJECT

Determination	Level Course		Surface Course							
	Untreated Sta. 124 + 50	Oil-Resin Emulsion Treated Sta. 123 + 50	Untreated				Oil-Resin Emulsion Treated			
			Sta. 119	Sta. 120	Sta. 122	Avg.	Sta. 119	Sta. 120	Sta. 122	Avg.
Penetration	56	113	68	70	63	67	91	100	123	105
Softening point	120	110	115	114	117	115	111	110	106	109

Surface Course	Untreated	Treated
Asphaltenes (% by wt.)	15.0	13.2
Nitrogen bases (% by wt.)	37.9	36.9
First acidaffins (% by wt.)	12.8	13.8
Second acidaffins (% by wt.)	22.1	23.3
Paraffins (% by wt.)	12.2	12.8
Maltenes dist. ratio, $(N + A_1)/(P + A_2)$	1.48	1.40

Samples of treated and untreated portions of this newly-constructed asphalt pavement were subjected to laboratory analysis. Table 6 summarizes the physical and mechanical properties of the surface course. The physical and mechanical properties of the asphalt concrete on the Panama Lane project were little affected by the addition of 0.1 gal per sq yd of the emulsion. The most noticeable effect was an increase in average asphalt content from 5.3 to 5.8 percent. The small differences in bulk specific gravity, stabilometer value, and cohesiometer value were well within the limits of reproducibility of the samples and the test procedures. Similar tests were not run on the asphalt concrete samples from the level course, because it was expected that the results would have followed the same pattern. Table 7 gives the results of tests on the asphalts recovered from the samples used to obtain the data in Table 6, together with recovered asphalts from a treated and untreated sample of level course material. The changes in properties of the asphalts were quite significant. Treatment with the emulsion increased the average penetration value of the asphalt by at least 38 points and lowered its average softening point by at least 6 F. Inasmuch as the original asphalt introduced at the pugmill was of grade 85 to 100, it is apparent that the treatment of 0.1 gal per sq yd was more than sufficient to restore the penetration points lost during the hot-mix cycle in the asphalt plant.

To verify results (Table 7) and obtain some indication of effect of treatment with depth, cores were taken one month later from the Panama Lane project in treated and untreated areas and the asphalt recovered. Table 8 gives the viscosity values, using the sliding plate microviscometer, determined on small samples of asphalt taken from the top, middle and bottom of each lift. Corresponding asphalt penetration values obtained from a conversion chart are also given.

The top portions of both the level and surface courses were significantly altered by the treatment with the

TABLE 8

TEST RESULTS ON CORES FROM PANAMA LANE TEST PROJECT

(Age: 1 Month)

Core No.	Location	Bulk Specific Gravity	Viscosity, Megapoises at 77 F, 5×10^{-2} SR					
			Surface Course			Level Course		
			Top	Middle	Bottom	Top	Middle	Bottom
U-1	125 + 00 OWT	2.22	2.7 (56)	2.8 (55)	2.1 (63)	1.8 (67)	1.3 (79)	1.6 (71)
U-2	125 + 00 BWT	2.18	4.8 (38)	3.8 (47)	2.3 (60)	3.3 (51)	1.4 (76)	1.0 (88)
U-3	128 + 75 OWT	2.21	--	--	--	--	--	--
U-4	128 + 75 BWT	2.20	--	--	--	--	--	--
T-1	119 + 00 OWT	2.20	--	--	--	--	--	--
T-2	119 + 00 BWT	2.14	--	--	--	--	--	--
T-3	123 + 00 OWT	2.22	0.13 (225)	1.8 (68)	1.7 (70)	0.65 (115)	0.97 (89)	0.99 (88)
T-4	123 + 00 BWT	2.20	0.64 (118)	2.5 (58)	2.8 (55)	1.1 (85)	1.3 (79)	1.2 (81)

Notes: Numbers in parentheses are penetration values from conversion chart. SR = strain rate; OWT = outer wheel track; and BWT = between wheel tracks.

TABLE 9

AIR AND WATER PERMEABILITY RESULTS, PANAMA LANE PROJECT, WESTBOUND LANE

Station	Water Permeability (ml/min)			Air Permeability for 4-In. Area (ml/min at ¼ in. V)		
	OWT	BWT	IWT	OWT	BWT	IWT
(a) Control Section						
124 + 50	--	135	--	--	879	--
125 + 00	85	--	--	471	--	--
+ 50	--	--	55	--	--	389
126 + 00	--	105	--	--	584	--
+ 50	100	--	--	628	--	--
127 + 00	--	--	77	--	--	565
+ 50	--	180	--	--	1,005	--
128 + 00	120	--	--	754	--	--
+ 50	--	--	40	--	--	502
129 + 00	--	190	--	--	1,633	--
+ 50	65	--	--	628	--	--
130 + 00	--	--	50	--	--	220
Avg.	93	153	61	622	1,027	421
Total Avg.		(102)			(691)	
(b) Cationic Maltenes Emulsion Test Section						
118 + 00	--	--	45	--	--	63
+ 50	--	40	--	--	471	--
119 + 00	50	--	--	251	--	--
+ 50	--	--	25	--	--	126
120 + 00	--	40	--	--	314	--
+ 50	40	--	--	170	--	--
121 + 00	--	--	28	--	--	170
+ 50	--	135	--	--	879	--
122 + 00	30	--	--	44	--	--
+ 50	--	--	47	--	--	60
123 + 00	--	90	--	--	295	--
+ 50	80	--	--	502	--	--
124 + 00	--	--	40	--	--	119
Avg.	50	76	37	242	490	107
Total Avg.		(54)			(280)	

Notes: Rolling consisted of breakdown with 12-ton tandem and 3 coverages with 25-ton pneumatic followed with 8-ton steel tandem. OWT = outer wheel track; BWT = between wheel tracks; IWT = inner wheel track.

oil-resin blend, whereas the middle and bottom portions were much less affected. Nevertheless, making allowances for variations in sampling and testing, the data indicate that the effects of the application are evident throughout the depth of the cores.

Another phase of the Panama Lane test project was to ascertain the effect of the emulsion on the permeability to both air and water of the newly-compacted asphalt pavements. Table 9 gives air and water permeability results from tests conducted on the pavement immediately after final compaction. All water permeability tests were run on the surface course using Calif. Test Method 341-A; the air permeability test method was that developed by the California Research Corporation (9).

The test section has had its water permeability reduced to about 53 percent of the untreated control section. The air permeability has been reduced to about 40 percent of the untreated control section. This is an important feature of the oil-resin emulsion treatment, particularly when the compaction effort required to reduce the permeability to a similar degree is considered. Moreover, the decrease in permeability was achieved in a finite depth of the pavement, and not by a skin coating which could readily be worn away by weather and traffic.

Evaluation of Skid Resistance

One of the problems associated with the application of any liquid material on a roadway is the question of its effect on the pavement's skid-resistant qualities. Because of the oily nature of the residue after evaporation of water, a hazard to traffic will exist under the following conditions:

1. If the emulsion is not fully absorbed by the pavement, due to excessive application, and remains at the surface;
2. If there is complete penetration of the emulsion but the oil-resin components can combine with accumulated grease drippings or remnants of previous asphalt seals at the surface; or
3. If exposed aggregate particles are smoothly polished and, therefore, easily lubricated.

In such cases, light sanding (1 to 2 lb per sq yd) with a dry and gritty material corrects the condition. However, such sanding should be delayed as long as possible, preferably a period of at least 45 min so that the emulsion has enough time to enter the pores of the pavement. If slick spots are localized, sanding should be confined to the specific areas. Problems are most likely to occur at intersections, sharp turns, and steep up-grades.

Table 10 gives coefficients of friction for two areas subjected to the emulsion treatment as compared to the same untreated surfaces. Complete penetration occurred in

TABLE 10

SKID RESISTANCE MEASUREMENTS WITH CALIFORNIA SKID TESTER

Location and Date	Section	Treatment	Degree of Penetration	Coefficient of Friction[1] Before: Dry	Before: Wet[2]	After: 1 Hr	After: 4 Hr	After: 28 Hr
Bear Mountain Blvd., July 13-14, 1960	R-1 Sta. 165 + 50	0.06 gsy O-R emul., 4:1	Residue at surface	0.41	0.32	0.29	0.28	0.30
	R-2 Sta. 193 + 50	0.08 gsy O-R emul., 2:1	Residue at surface	--	0.34	0.28	0.27	0.31
	R-3 Sta. 221 + 50	0.07 gsy O-R emul., 1:1	Residue at surface	0.41	0.32	0.28	0.28	0.33
Meadows Field, July 14, 1960	Control	None	--	0.43	--	--	--	--
	I	0.11 gsy O-R emul., 1:1	Complete in 3 min	--	--	--	--	0.41
	II	0.22 gsy O-R emul., 1:1	Complete in 5 min	--	--	--	--	0.41

[1]All values average of 5 or more tests made at 50 mph using smooth tire.
[2]With glycerine.

one instance, but a residue remained in the other because of the relative imperviousness of the asphalt concrete. A loss in frictional resistance results when penetration is not complete. Although sufficient frictional resistance remains for traffic to negotiate the area, the traveling public generally cannot be expected to respond properly to such a condition particularly at high rates of speed. Therefore, sanding is recommended as a precautionary measure.

Where penetration of the emulsion is complete, the coefficient of friction remains high and no sanding is required, although it is recommended at intersections and sharp turns as a precautionary measure.

For areas that can be restricted to traffic for long periods of time, it is beneficial to allow the treatment to soak in without any sanding.

CONCLUSIONS

On the basis of the information and data presented, together with the experience gained over the past three years of application under many and varied conditions, the following conclusions are made:

1. The principle of rejuvenating aged asphalts, in situ, with a selected combination of petroleum oils and resins is a sound and workable one.
2. The best procedure for introducing the oil-resin fraction into the pavement and carrying it to the existing asphalt films without displacing them or destroying their cohesiveness is by use of an emulsion.
3. A cationic charge on the emulsion is necessary to effect preferential wetting of the asphalt over the aggregate by the effective ingredients of the emulsion.
4. The simple "dilute, spray, and penetrate" procedure works well as a construction seal on new pavements and in tack or prime coating operations in preventive maintenance treatment of relatively new pavements (2 to 10 years old).
5. Treatment of dense, old, badly-cracked or tightly-sealed asphalt pavement must be done in combination with other procedures, such as heating, planing, discing, scarifying or mixing with the cationic maltenes emulsion applied during the operation to bring the weathered asphalt and the emulsion into intimate contact.
6. When the emulsion and weathered asphalt are exposed to each other, data from both laboratory experiments and large-scale field tests indicate that the properties of the aged asphalt are significantly improved with respect to consistency, adhesiveness, and durability without any detrimental effects on the cohesiveness or stability of the pavement.
7. Combining the oil-resin blend with the aged asphalt causes an expansion of the asphalt which blocks the pores and results in a significant decrease in permeability to both air and water.

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Bronze plug, 1.128 in. diameter approximately two inches long, and clip to fasten plug securely to center of upper platen of Unconfined Compression Tester.

Two Stopwatches.

Source of Compressed air, 125 psi minimum.

Materials

Asphalt briquets, as described in Appendix A, treated and/ or aged as specified.

Paper squares, approximately 4x4 in.

Calibration

Adjust rate-of-strain control valve to give a rate, unloaded, of 0.25 $\pm$ 0.02 in. per minute at average available air pressure.

Procedure

Convenient operation requires two people, one to operate tester and call out strain readings while the second reads and records stress readings. Place briquet on a paper square on the lower platen of tester; center carefully. Depress main control valve. Start both stopwatches when strain and/or stress gauges first begin steady climb. Record stress readings at specified strain readings and at maximum stress. Stop first watch at point of maximum stress; stop second watch at end of test (0.25 in. strain, but do not exceed maximum allowable stress on proving ring).

Note: With the pneumatic-operated hydraulic tester, the actual rate of strain varies from about 0.25 in. per minute for relatively weak briquets to 0.1 in. or even less per minute on very strong briquets, due to the slowing effect of the resistance on the travel of the piston, and to greater movement of the proving ring. An improved tester is now being designed, utilizing a constant speed drive and an electrical load measurement device which eliminates motion within the load measuring mechanism.

Appendix C

METHOD OF TEST FOR DETERMINING THE QUANTITY AND RATE OF ABSORPTION OF RECLAMITE INTO AN ASPHALT PAVEMENT

Equipment

(1) Watch with second hand, preferably a stop watch.

(2) Six inch diameter template.

(3) Piece of yellow lumber crayon or chalk.

(4) Grease gun or calking gun or commercially available grease-filled plastic tube capable of slowly extruding medium chassis grease in a continuous ribbon approximately 1/4 inch diameter.

(5) Supply of medium chassis grease (for grease gun or calking gun only).

(6) Spatula or putty knife.

(7) Quart can containing a dilution of 2 parts Reclamite to 1 part water (2:1 dilution).

(8) 25 ml plastic or glass graduate (plastic preferred).

(9) Small brush with stiff bristles.

(10) Empty open-end pint can for waste grease.

(11) Quart can of water.

(12) Several rags.

Procedure

(1) Using 6 in. diameter template and crayon or chalk circumscribe circle on asphalt pavement where test is to be run.

(2) With grease or calking gun or grease-filled plastic tube, place a 1/4 inch (approximately) bead of grease on the circumference of the circle.

(3) Run index finger around outside edge of grease ring making sure to push a small amount of grease tightly against the pavement. This will form a sealed reservoir for the test solution.

(4) Measure 8.3 ml of Reclamite dilution (2:1) in graduate and pour in grease ring, simultaneously starting stop watch or re-

cording time to nearest second.

Note: 4.1 ml equivalent to 0.05 gsy spread rate

8.3 ml	"	" 0.10 gsy	"	"
16.5 ml	"	" 0.20 gsy	"	"
24.8 ml	"	" 0.30 gsy	"	"

(5) Using small brush quickly spread Reclamite dilution uniformly over area of circle.

(6) Clean graduate by rinsing with water.

(7) Record time interval required for Reclamite dilution to penetrate into surface.

Note: Complete penetration is generally indicated by loss of pink color of Reclamite dilution, except when an extended time of penetration allows evaporation of water with subsequent breaking of Reclamite emulsion on surface. Latter possibility readily evident because of tacky film of Reclamite residue on surface.

(8) After test is completed scrape up grease ring with spatula and place in open-end pint can.

(9) If 8.3 ml is absorbed within a 15-minute interval, make new grease ring and repeat test with additional testing fluid in increments of 8.3 ml (16.5 ml, 24.85 ml, etc.) until time of penetration exceeds 15 minutes.

(10) If 8.3 ml is not absorbed within a 15-minute interval, repeat test using 4.1 ml of testing fluid.

Test Report

The following information and data should be recorded:

(1) Description of surface being tested.

(2) Location of test ring.

(3) Time of penetration for each quantity of Reclamite dilution used.

(4) Estimate of quantity of Reclamite dilution absorbed by pavement in 15 minutes.

Note: The test rings should be examined 24 hours later to determine visually the effectiveness of the treatment. Test area can also be probed with a knife blade or screwdriver to qualitatively determine depth of penetration.

Aggregate Degradation in Bituminous Mixtures

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A laboratory study was performed using a gyratory testing machine to determine the factors affecting the degradation of aggregate in bituminous mixtures.

Three kinds of aggregates with different Los Angeles values were used. The aggregates were blended according to three different gradations ranging from an open gradation to a Fuller maximum density gradation. Four different asphalt contents were used. Use of a gyratory testing machine made it possible to change the compactive efforts in two different ways: change in magnitude of load and change in repetition of load.

The results of this study indicated that, regardless of type of aggregate, gradation, compactive effort, method of compaction, and presence of asphalt, each fraction of aggregate degraded in such a way that its sieve analysis curve was a smooth curve approaching a parabola, which implied that the pattern of degradation is constant. The magnitude of degradation, as measured by percent increase in surface area, was found to vary and to depend on the foregoing variables. Gradation was found to be the most important factor affecting degradation; the denser the mix the less the degradation. Soft aggregate with a high Los Angeles value degraded less than hard aggregate with a low Los Angeles value when the former was blended in a dense mixture and the latter in an open mixture.

Degradation also varied with type of aggregate. In general, aggregates with high Los Angeles values resulted in more degradation than those with low Los Angeles values. The rocks with good interlocking and strong cementation between the grains produced less degradation than rocks with loose interlocking and weak cementation. Increase in compactive effort, either by increase in magnitude of load or by increase in number of repetitions of load, increased the degradation. However, the magnitude of load was found to effect degradation more than repetition of load. The effect of asphalt was found to be dependent on other factors, and there was no definite pattern for the effect of asphalt content on degradation without considering other variables.

• A BITUMINOUS mixture is essentially a three-phase system consisting of bitumen, aggregate and air. In order for such a mixture to serve its purpose, it is compacted

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to a certain degree during construction. During its life, the mixture is subjected to further compaction due to the action of traffic. This further densification of a bituminous mixture under traffic may produce progressive deterioration of the pavement, either by reduction of voids to the point where a plastic mixture results, or by producing raveling. In either case, degradation of the aggregate may play an important role.

Compaction is an energy-consuming process, which results from the application of forces to the mixture. The mixture withstands these forces in many ways, such as by interlock, frictional resistance, and viscous or flow resistance. When the applied forces have a component in any direction greater than the resistance of the mat, the material will move and shift around until a more stable position is attained. This rearrangement of the material, especially the aggregate phase, causes a closer packing of particles, a new internal arrangement or structure, and a higher unit weight.

The energy required for the relocation or rearrangement of particles is provided by contact pressure, and the particles while adjusting to their new locations are subjected to forces which cause breakage and wear at the points of contact. This phenomenon, called degradation, reduces the size of particles and changes the gradation of aggregate which in turn causes a reduction in void volume and an increase in density. Any change in the gradation of the aggregate in a mix causes an associated change in basic properties of the bituminous mixture, namely, stability and durability. In some mixtures the change of gradation due to degradation of aggregate causes the asphalt present in the voids to be pushed out and an unstable mix to result, whereas in other mixtures the amount of asphalt present is not sufficient to coat all the newly produced surfaces and disintegration of the mat results. Degradation may reduce the angularity of aggregate particles, and thus decrease the interlocking which in turn results in a loss in stability of the mixture with resulting shoving, distorting, and corrugating.

A review of the literature concerning aggregate qualities shows that numerous methods have been used to determine the suitability of aggregates for road-building purposes. The most important objective of these methods is to provide reliable criteria for accepting or rejecting aggregates. In general the properties of aggregate can be divided into two classes: (a) those belonging to the individual piece and (b) those belonging to the aggregation of pieces. Obviously, the properties of the second group depend on the properties of the first. Determination of the relationship between properties of the two classes may not be difficult when the pieces are all identical, but it is a difficult problem when the pieces exist in a range of sizes, a variety of shapes, and sometimes even have different compositions. The most important tests among the first group are those concerned with petrographic analysis and with impact and crushing strength. Among the second group, tests such as abrasion tests, compression tests, and field and laboratory roller tests, are pertinent to the degradation of aggregates, especially in bituminous mixtures.

The relative importance of factors affecting the degradation of aggregates is generally a matter of controversy among investigators. Factors such as type of aggregate, maximum size and gradation of particles, aggregate shape, asphalt content, and compactive effort are all cited in the literature as controlling factors of aggregate degradation.

The purpose of this investigation was to evaluate the degradation characteristics of aggregates in bituminous mixtures and to analyze the factors which are effective in causing this degradation. In so doing, the following factors were investigated: (a) type of aggregate, (b) gradation of aggregate, (c) aggregate shape, (d) aggregate size, (e) asphalt content, and (f) compactive effort.

MATERIALS AND PROCEDURE

Three kinds of aggregates were used in this study—dolomite, limestone and quartzite. Selection was based on a relatively wide range of Los Angeles values and on petrographic structure. Table 1 gives data on origin, specific gravity, Los Angeles value, and compressive strength. Table 2 summarizes petrographic analysis results.

An 85- to 100-penetration grade asphalt cement was used in this study; test results are given in Table 3.

TABLE 1

RESULTS OF LOS ANGELES ABRASION AND COMPRESSIVE STRENGTH TESTS[a]

Grading or Specimen Size	Dolomite	Limestone	Quartzite
(a) Los Angeles Abrasion			
Grading[b]:			
A	40.0	26.7	22.0
B	41.0	25.0	23.7
C	33.0	27.5	24.9
(b) Compressive Strength (psi)[c]			
Specimen size (in.):			
1.0 × 1.0 × 1.0	10,100	15,000	25,200
1.0 × 1.0 × 2.0	8,500	14,300	29,600

[a]Each value is the average of three tests.
[b]According to ASTM Method C 131.
[c]Rate of loading 0.025 in./min.

TABLE 2

PETROGRAPHIC ANALYSIS

Determination	Dolomite	Limestone	Quartzite
Megascopic identification	Dolomite, medium-grained, indistinct banding	Calcite, medium-grained, indistinct banding	Hematitic, medium-grained quartzite, indistinct banding, numerous recemented fractures
Bulk minerals:			
Kind	Dolomite Fine pyrite	Calcite Pyrite Organics	Quartz Pyrite
Volume (%)	99 1	>95 1-2 1	90 4-7
Avg. grain size (mm)	0.2 -	0.5 0.2 -	0.8 0.1
Range (mm)	0.1-0.4 -	0.1-1 0.1-0.3 -	0.01-1.0 -
Composition and nature of matrix and cementing material	Smaller mesh of dolomite	Fine-grained carbonate matrix	Very fine-grained quartz and sericite (fibrous)
Decomposition	Nil	Nil	Nil
Degree of leaching	Minor	Nil	Nil
Secondary minerals	Negligible, where present consist limonite and hematite	Total % (vol.) 1 limonite, hematite	Hematite as coatings and finely disseminated grains; sericite in seams and disseminated throughout
Secondary cementation	Absent	Unobservable	-
Percent void	6.0	0.7	0.5
Nature of the grain boundaries	Loose interlocking	Good interlocking	Rock and grains are both highly fractured (cataclastic structure); all quartz grains display a prominent wave extinction, indicating a highly-stressed rock
Fracturing and cracking	Low	Not significant	-
Particle orientation	Random (sometimes lineation due to deposition)	Random	Moderate lining along the long axis of the grains
Banding	Indistinct	Indistinct banding; lenses of fine particles	Moderate banding depending on particle size
Other structure	Several pockets with concentration of very fine-grained materials; low porosity in pockets	Marked change from very coarse mesh to very fine mesh	Recemented granulated matrix

TABLE 3

RESULTS OF TESTS ON ASPHALT CEMENT

Specific gravity, 77/77 F	1.032
Softening point, ring and ball (°F)	114.0
Ductility, 77 F (cm)	200+
Penetration (mm):	
100 g, 5 sec, 77 F	90
100 g, 5 sec, 32 F	20
Flash point, Cleveland open cup (°F)	600
Solubility in CCl_4 (%)	99.8

TABLE 4

ORIGINAL GRADATIONS

Sieve	Analysis (% passing)		
	Grading O	Grading B	Grading F
½ In.	100.0	100.0	100.0
⅜ In.	75.0	86.0	86.6
No. 3	50.0	62.0	70.7
No. 4	25.0	50.0	61.2
No. 6	0.0	45.0	51.4
No. 8	--	36.0	43.3
No. 12	--	25.0	36.3
No. 16	--	16.0	30.0
No. 30	--	11.0	22.0
No. 50	--	6.0	15.0
No. 100	--	4.0	10.9
No. 200	--	3.0	7.7

The three gradations (Table 4 and Fig. 1) ranged from an open grading, consisting only of the top four sizes, to a Fuller gradation for well-graded material. The maximum size of all three gradations was one-half inch.

Aggregates for each specimen were batched by component fractions according to the blend formula. A batch consisted of 1,000 g. The blended aggregates for specimens containing asphalt were heated to 275° ± 10°F. The asphalt was heated separately

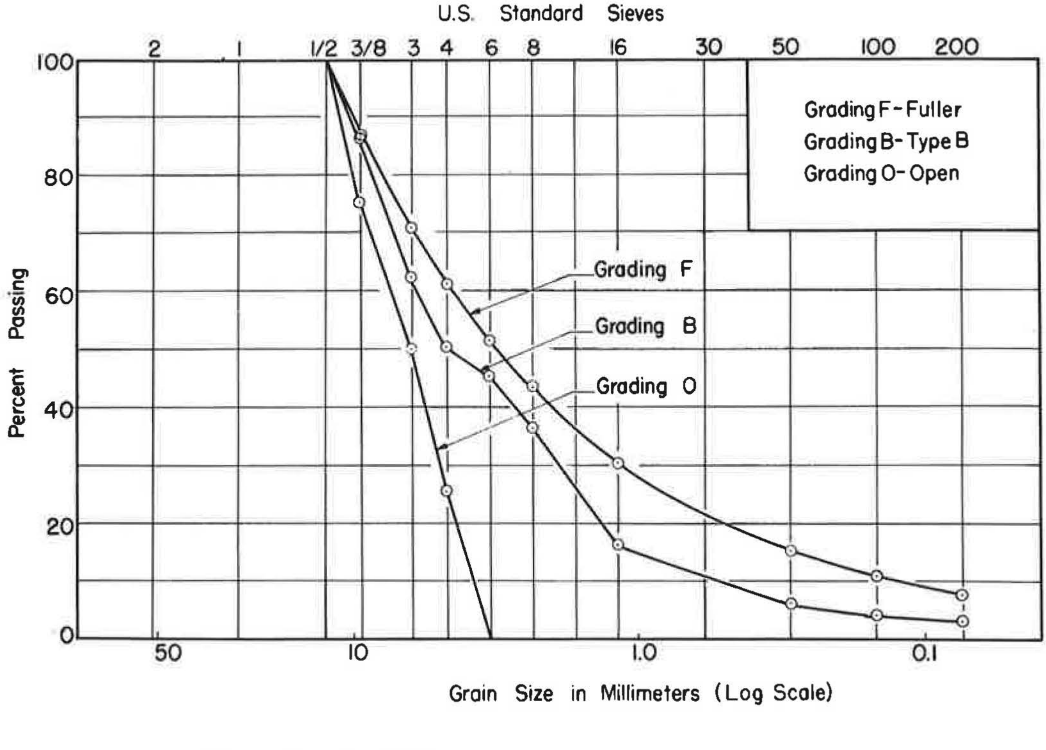

Figure 1. Gradation curves for original gradations.

to 290° to 300°F. Mixing was for 2 min using a Hobart electric mixer modified with a special paddle and a scraper. For those cases in which the aggregate was tested without asphalt, the aggregate was not heated or subjected to mixing.

Because this study was solely a laboratory investigation, a fundamental part of it was the selection of testing equipment which would produce specimens similar to pavement with respect to density and structure. Many methods of compaction have been devised and used to simulate field compaction in the laboratory. Most methods are based principally on the concept of equal density. Equal density without regard to orientation and degradation of particles cannot produce representative specimens and, unfortunately, there is no way to measure the structure of specimens quantitatively. The only way in which it seems possible to compare the structure of the compacted materials is to compare the forces involved in producing the laboratory specimen and the field mat. The methods that incorporate horizontal forces and apply shear to the specimen throughout its depth seem to be the most suitable ones. Therefore, of all available methods, gyratory compaction appeared to be the most promising one to produce specimens similar to the field mat from the density and structure standpoint.

A gyratory testing machine (Fig. 2) was used. It was possible to change the compactive effort in two ways: (a) change in magnitude of load, and (b) change in repetition of load. The magnitude of load, controlled by vertical pressure, was varied from 50 to 250 psi; and the repetition of load, controlled by the number of gyrations, ranged from 30 to 250, for the most part, but in some cases up to 1,000 gyrations were used.

The mixtures were brought from the mixing temperature to 230°F and were placed in the gyratory machine for compaction. Electric heating elements around the mold provided an elevated temperature throughout the test. After gyration, an extraction test was made on the whole specimen, and the gradation of the extracted aggregate was determined for comparison with gradation before mixing and compaction.

To study the effect of particle shape on degradation, it was desirable that the rounded pieces not differ from the crushed ones in composition. Therefore, artificially rounded pieces were produced by subjecting angular pieces to a few thousand revolutions in a Los Angeles machine (Fig. 3).

To investigate how various sizes of aggregate degrade in an aggregation of pieces of different sizes, the three top sizes were dyed different colors so that after compaction and extraction of asphalt the newly produced pieces could be associated with the original piece by colored faces. For this purpose the dyes had to be soluble in water, stay on the surface of the piece, and not be soluble in asphalt or the trichloroethylene used in extraction. The following dyes were found to have such characteristics: (a) orseillin BB red, (b) crystal violet, (c) malachite green oxalate.

Figure 2. Gyratory testing machine.

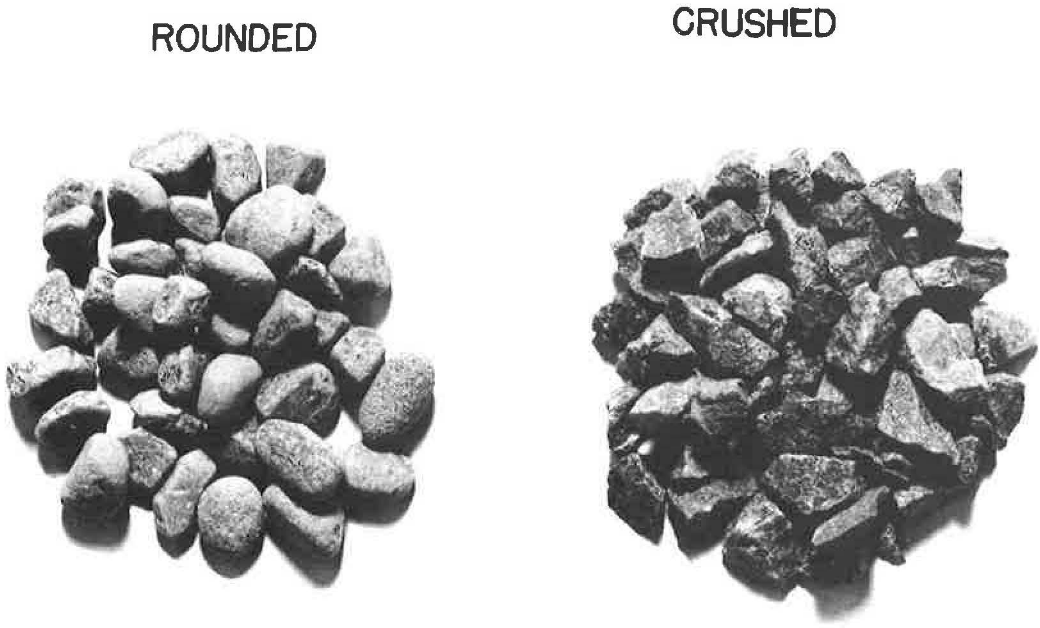

Figure 3. Rounded and crushed quartzite.

RESULTS

Of the several methods available to represent the degradation characteristics of aggregate, two were chosen: one was a simple gradation curve of percent smaller than certain sizes, and the other was based on surface-area concepts. Using the surface-area concept, measurements of the degradation were made on the basis of surface-area increase as determined by sieve analysis. The computing factors are given in Table 5 for an assumed specific gravity of 2.65. Values were calculated on the assumption that all material passing the No. 4 sieve was spherical and that retained was one-third cubes and two-thirds parallelepipeds with sides of 1:2:4 proportions.

It was decided that numerical increase in surface area, which is merely the difference between the final surface area and the original surface area, is not a satisfactory measure of aggregate degradation. For example, when a mixture with an original surface area of 2.2 cm^2/g has increased 2.2 cm^2/g in surface area after compaction, and another mixture with 67.3 cm^2/g has increased the same amount, it cannot be considered that the two mixtures have undergone equal degradation. The first mixture has gained 100 percent in surface area (its final surface area is twice the original), whereas the second mixture has increased only 3 percent.

TABLE 5

SURFACE-AREA FACTORS[a]

Fraction of Material		Factor (sq cm per g)
Passing	Retained	
½ In.	⅜ In.	2.2
⅜ In.	¼ In.	3.2
No. 3	No. 4	4.5
No. 4	No. 6	5.7
No. 6	No. 8	7.9
No. 8	No. 16	12.7
No. 16	No. 50	30.0
No. 50	No. 100	100.0
No. 100	No. 200	205.0
No. 200	Pan	615.0

[a]Assumed specific gravity = 2.65; for values other than 2.65, multiply the above factors by 2.65/sp. gr.

Therefore, it was decided to express the data in percent increase in surface area rather than increase in surface area. Another advantage of the percentage method is the elimination of the necessity for correction of surface-area values for specific gravity.

Herein, the term degradation is used to include all of the aggregate breakdown due to mechanical action regardless of the type of mechanical action. Degradation can result from aggregate fracture or breakage through the piece, from chipping or corner breakage, and from the rubbing action of one piece or particle against another. In parts of this study, attempts were made to separate degradation into two parts: one due to fracture through the piece and designated as breakage, and the other due to corner breakdown and attrition which collectively have been designated as wear.

Degradation of One-Sized Aggregate

Size of particles and maximum size of particles are cited in the literature among the factors controlling degradation. To determine whether or not change of size will change the degradation characteristics of an aggregate and to investigate the effect of combinations of pieces of different sizes on degradation, specimens of one-sized aggregate were tested (Table 6).

Figure 4 shows the sieve analysis on specimens made of limestone aggregate; regardless of size of aggregate, all the curves approach a parabolic shape. Plots of the data in Table 6 for the other two aggregates would show that this conclusion obtains with respect to type of aggregate as well. As original size of particles decreases there is also a corresponding increase in fine material, which suggests that degradation increases as particle size decreases. Figure 5 shows the percent increase in surface area versus average size of original particles for the three aggregates. As the size of one-sized aggregate increases, the degradation under equal compactive effort (200 psi and 100 revolutions) increases.

At first glance it appears that the results of sieve analysis and percent increase in surface area are in conflict. However, sieve analysis indicates only what percent of material is of which size, without considering its original condition and the changes through which it has gone. A larger piece has to undergo more breakdown than a smaller particle to be reduced to a certain size. Therefore, although sieve analysis is an excellent way to study the pattern of degradation, it is by no means a measure of degradation. By relating the produced area to the original area, the concept of percent increase in surface area is a much better means of measuring degradation.

Figure 5 also shows that degradation increases from quartzite to limestone to dolomite, following the same pattern indicated by the Los Angeles rattler test. In other words, degradation of one-sized material increases as it becomes weaker and softer (higher Los Angeles value).

TABLE 6

RESULTS OF GYRATORY TESTS OF VARIOUS ONE-SIZED AGGREGATES
(200 psi; 100 revolutions)

Determination	Dolomite				Limestone				Quartzite			
Original Size	1/2 to 3/8 In.	3/8 In. to No. 3	No. 3 to 4	No. 4 to 6	1/2 to 3/8 In.	3/8 In. to No. 3	No. 3 to 4	No. 4 to 6	1/2 to 3/8 In.	3/8 In. to No. 3	No. 3 to 4	No. 4 to 6
(a) Total Percent Passing												
Sieve size:												
1/2 in.	100.0	--	--	--	100.0	--	--	--	100.0	--	--	--
3/8 in.	59.8	100.0	--	--	55.3	100.0	--	--	48.6	100.0	--	--
No. 3	37.3	53.6	100.0	--	32.0	58.4	100.0	--	23.2	43.8	100.0	--
No. 4	30.6	37.4	48.5	100.0	24.9	34.1	54.3	100.0	17.9	26.6	37.0	100.0
No. 6	25.2	29.6	32.5	46.5	20.2	25.7	33.7	53.6	14.0	19.2	19.3	38.1
No. 8	21.3	24.5	25.8	31.0	16.5	18.8	24.7	32.3	11.3	14.8	14.5	20.8
No. 16	14.2	16.4	16.7	18.7	10.7	12.1	14.7	17.0	7.0	8.8	8.3	10.6
No. 50	7.2	8.1	8.4	9.0	4.7	4.8	5.8	6.2	3.1	3.5	3.4	3.7
No. 100	5.4	6.0	6.1	6.8	2.9	3.1	3.6	3.8	1.8	2.1	2.2	2.4
No. 200	3.8	4.1	4.5	5.0	1.8	2.0	2.2	2.4	1.1	1.3	1.5	1.6
(b) Weight and Surface Area												
Total weight (g)	1,000.0	1,000.0	1,000.5	1,000.0	992.5	992.5	993.0	1,000.0	1,000.0	1,000.0	1,000.0	1,000.0
Surface area:												
Final (cm²/g)	34.0	37.8	40.4	45.5	19.7	22.6	25.9	29.6	13.8	16.9	18.4	20.1
Original (cm²/g)	2.2	3.2	4.5	5.7	2.2	3.2	4.5	5.7	2.2	3.2	4.5	5.7
Increase (cm²/g)	21.8	34.6	35.9	39.7	17.5	19.4	21.4	23.9	11.6	13.7	13.9	14.4
Increase (%)	1,443.0	1,081.0	800.0	696.0	795.4	606.2	479.0	419.3	528.6	428.1	308.9	252.6

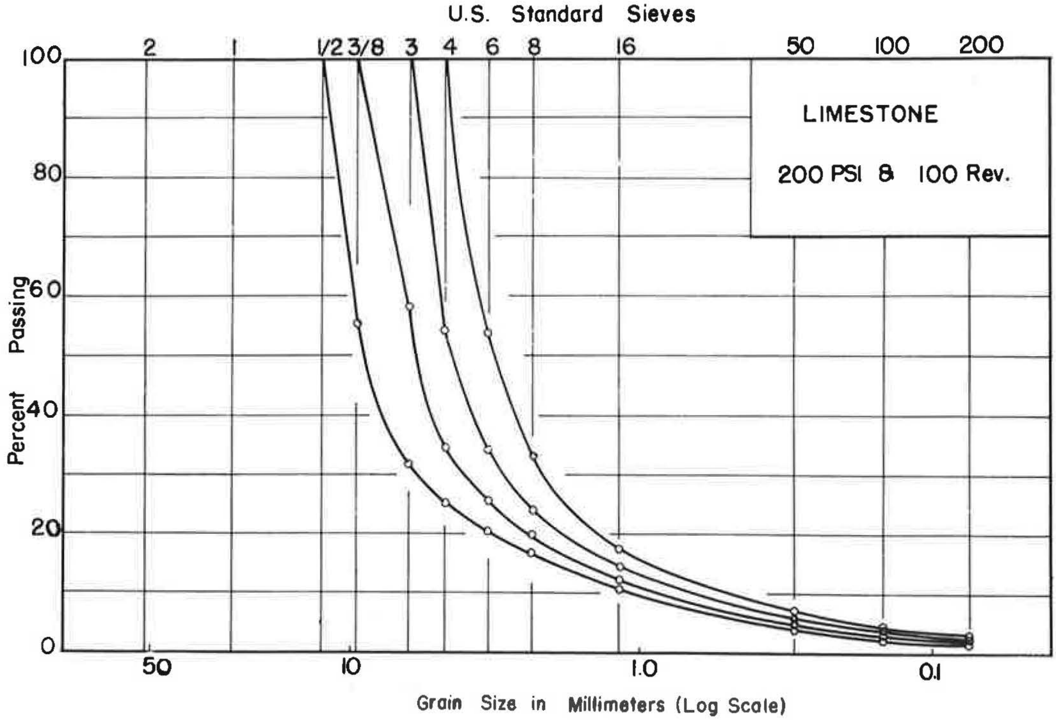

Figure 4. Sieve analysis of one-sized limestone aggregates after gyratory compaction.

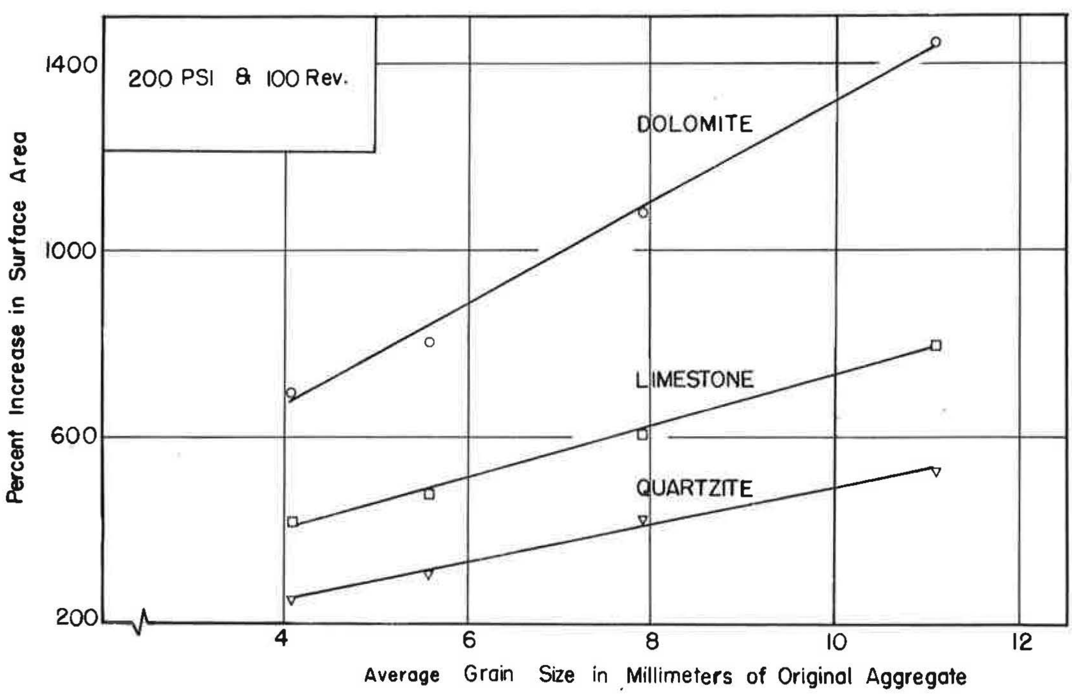

Figure 5. Degradation vs aggregate size; gyratory compaction, one-sized aggregate.

Figure 6 shows a linear relationship between the Los Angeles values of the three kinds of aggregate and the degradation of the one-sized aggregate when tested in the gyratory compactor and measured in percent increase in surface area.

The effect of change of compactive effort on the degradation of one-sized aggregate was studied by changing the number of revolutions of gyratory compaction. Five specimens of each kind of aggregate (original sieve size: $^3/_8$ in. to No. 3) were compacted under 100-psi ram pressure and five different numbers of revolutions (Table 7). The results of sieve analysis of dolomite aggregate after compaction indicate that the general shape of the gradation curve is not changed by a change in compactive effort; as compactive effort increases the curve shifts upward (Fig. 7).

Figure 8 shows that as compactive effort increases the degradation also increases, but generally a significant portion of the degradation occurs under the first few hundred revolutions and then the curves start leveling off. As the material becomes softer or weaker, the slope of the latter part of the curves increases, indicating that the degradation of such materials is more susceptible to change in compactive effort.

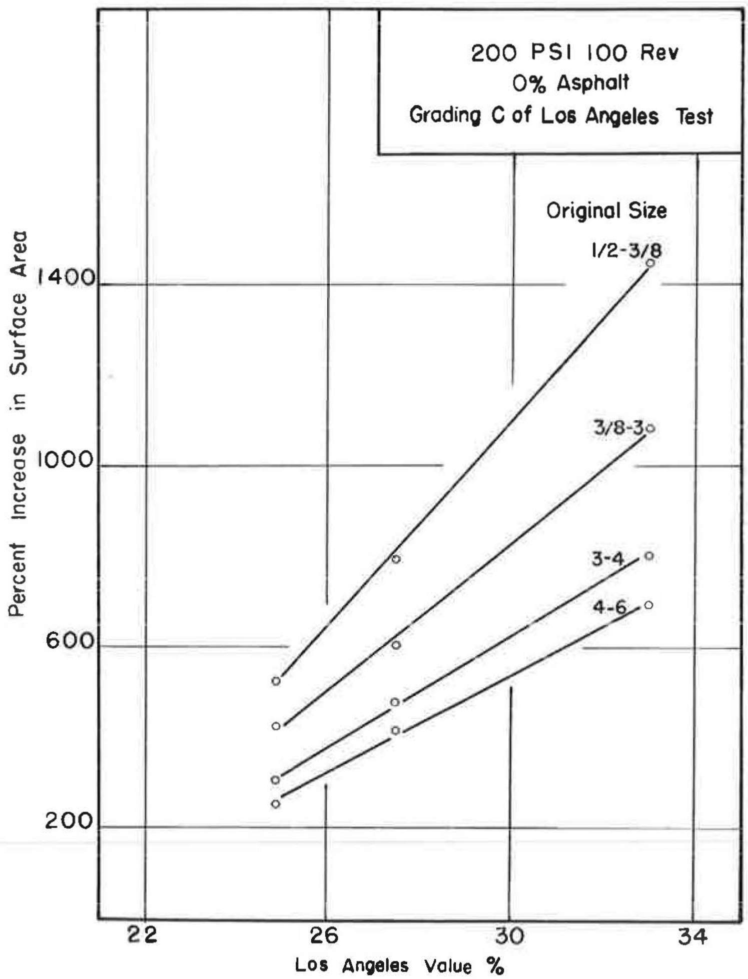

Figure 6. Degradation vs Los Angeles value; gyratory compaction, one-sized aggregate.

TABLE 7

RESULTS OF GYRATORY TESTS OF ONE-SIZED AGGREGATES

(100 psi)

Determination	3/8 In. to No. 3 Dolomite					3/8 In. to No. 3 Limestone					3/8 In. to No. 3 Quartzite				
No. of Rev.	50	100	250	500	1,000	50	100	250	500	1,000	50	100	250	500	1,000
(a) Total Percent Passing															
Sieve size:															
3/8 in.	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
No. 3	34.3	42.9	45.6	48.2	50.0	29.7	36.7	38.9	41.9	47.3	28.8	31.6	35.7	39.8	43.7
No. 4	19.5	22.0	25.8	30.7	32.1	16.8	21.2	23.3	27.4	30.5	15.2	17.1	20.0	23.8	27.5
No. 6	14.5	16.0	20.3	23.0	25.5	11.0	14.8	16.6	20.1	23.1	10.4	11.9	14.3	17.8	20.7
No. 8	11.1	12.5	15.5	18.5	20.6	8.5	11.6	13.4	16.6	19.8	7.6	8.8	10.9	14.0	16.5
No. 16	6.6	7.3	10.0	14.0	15.6	4.6	6.6	8.2	10.7	13.4	4.1	4.9	6.4	9.0	10.8
No. 50	3.2	3.6	5.5	7.3	8.3	1.8	2.7	3.5	4.7	6.2	1.5	1.9	2.6	4.1	4.9
No. 100	2.4	2.7	4.1	5.5	6.1	1.2	1.8	2.3	3.1	4.1	0.9	1.1	1.6	2.4	3.0
No. 200	1.7	2.0	2.8	3.9	4.3	0.8	1.3	1.5	2.1	2.7	0.5	0.7	1.0	1.5	1.8
(b) Weight and Surface Area															
Total weight (g)	1,000.0	999.5	1,000.0	1,000.0	1,000.0	1,000.0	1,000.0	995.0	1,000.0	1,000.0	1,000.0	999.5	999.0	1,000.0	1,000.0
Surface area:															
Final (cm^2/g)	18.0	19.8	27.2	34.0	38.3	11.5	15.3	17.8	22.4	27.9	9.6	11.3	13.7	18.3	21.2
Original (cm^2/g)	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
Increase (cm^2/g)	14.8	16.6	24.0	30.8	35.1	8.3	12.1	14.6	19.2	24.7	6.4	8.1	10.5	15.1	18.0
Increase (%)	463.0	530.0	750.0	962.0	1,097.0	260.0	378.0	457.0	600.0	773.0	200.0	255.0	330.0	473.0	563.0

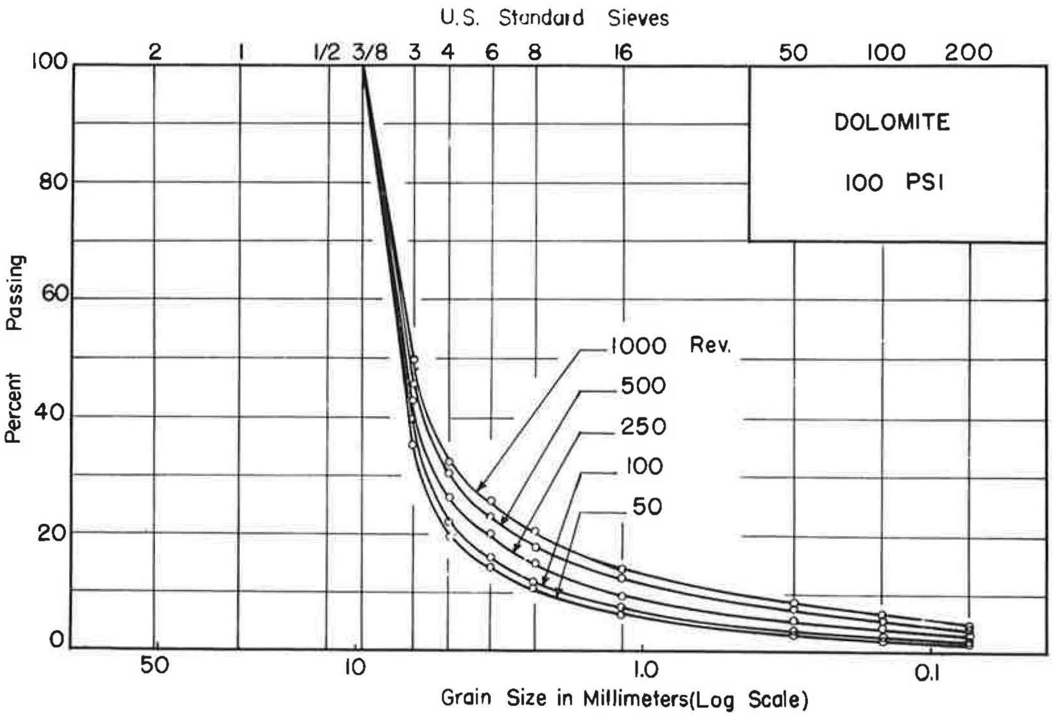

Figure 7. Sieve analysis of one-sized dolomite aggregates; varying number of revolutions of gyratory compactor.

Degradation of Individual Sizes in an Aggregation of Sizes

Before making a detailed analysis of the effect of variables on degradation of different mixtures, it was necessary to investigate the changes that occur in degradation characteristics of each particle size due to the presence of other sizes in the specimen. A dyeing process was used to determine the size fraction from which each particle was produced when degradation occurred. Because previous studies indicated that the kind of aggregate changes only the magnitude of degradation and has no effect on its pattern, it was decided to use only one kind of aggregate: the limestone, with the intermediate Los Angeles value, which could be satisfactorily dyed. Because separating

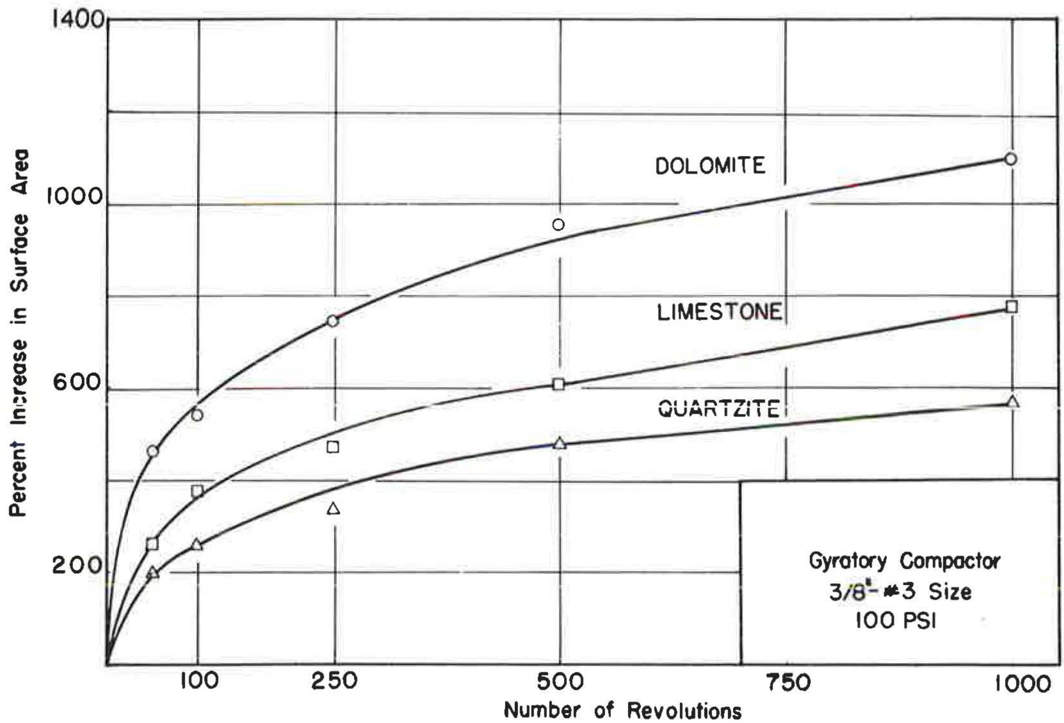

Figure 8. Degradation vs number of revolutions for one-sized aggregates.

the fractions of different colors by hand is time-consuming, it was decided to dye only the top three sizes—½ in. to ⅜ in., ⅜ in. to No. 3, and No. 3 to No. 4. Had a difference in pattern of degradation due to the size been noticed, then other sizes would have been dyed. The materials were separated only down to the No. 30 sieve. The factors considered as variables were gradation of aggregate, compactive effort, and presence or absence of asphalt.

The gradings O, B, and F given in Table 4 were used. Twenty-four samples of three gradations, without asphalt and with 4 percent asphalt, were tested under four different compactive efforts in the gyratory machine. The results of sieve analysis of each colored fraction, and sieve analysis of the total specimen are given in Tables 8, 9, and 10.

Figure 9 shows the sieve analysis of each fraction of a specimen without asphalt having an original open gradation and being subjected to 200-psi ram pressure and 100 revolutions in the gyratory compactor. The curves indicate that the degradation of each fraction has a constant pattern of a smooth curve approaching a parabola. Figures 10, 11, and 12 show the sieve analysis of each fraction for specimens with 4 percent asphalt and original gradings O, B, and F. The pattern of degradation of each fraction is also a constant.

The results obtained with colored aggregate showed that when particles of different sizes are mixed together and subjected to a certain compactive effort each size will break down into smaller particles whose new gradation has a characteristic size distribution. This size distribution follows a smooth curve and approaches a parabola similar to the curves obtained for specimens made of one-sized aggregates tested separately. Therefore, this portion of the study indicated that degradation of one-sized particles follows a definite pattern regardless of its size or the gradation with which it is associated, magnitude of compactive effort, or presence of asphalt. Also, from the first part of the study it was found that the degradation pattern is independent of kind of aggregate; hence, it can be concluded that when the pattern of degradation of

TABLE 8

RESULTS OF SIEVE ANALYSIS OF COLORED AGGREGATES, GRADING O_1

Sieve	Total Percent Passing									
	30 Rev.					100 Rev.				
	½ to ⅜ In. Violet	⅜ In. to No. 3 Red	No. 3 to 4 Green	No. 4 to 6 Natural	Total	½ to ⅜ In. Violet	⅜ In. to No. 3 Red	No. 3 to 4 Green	No. 4 to 6 Natural	Total
(a) 100 Psi; 0% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	25.5	100.0			81.4	27.7	100.0			83.3
No. 3	11.6	24.3	100.0		59.0	13.2	32.4	100.0		61.0
No. 4	8.2	10.0	31.1	100.0	37.3	10.0	14.7	40.4	100.0	40.8
No. 6	5.6	7.0	15.0	48.2	18.4	7.0	10.3	22.1	59.8	24.5
No. 8	4.0	5.2	10.4	23.7	10.8	5.2	8.1	15.5	32.4	15.2
No. 16	1.9	3.2	5.3	9.6	5.0	3.2	5.0	8.0	15.6	7.9
No. 50	0.9	1.6	2.0	2.2	1.6	2.0	2.5	3.0	4.0	2.8
No. 100					1.0					1.8
No. 200					0.7					1.1
Total Weight, g	250.0	251.0	251.0	251.0	1,003.0	251.5	251.5	251.5	251.5	1,006.0
(b) 200 Psi; 0% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	44.0	100.0			86.0	52.2	100.0			88.1
No. 3	19.4	45.6	100.0		66.8	23.6	49.4	100.0		68.3
No. 4	14.0	20.5	43.0	100.0	44.7	16.6	22.2	49.4	100.0	47.1
No. 6	10.8	13.9	24.5	69.1	29.6	12.8	16.4	28.4	77.2	33.7
No. 8	8.6	10.9	16.9	39.8	19.1	10.2	12.6	20.8	48.8	23.1
No. 16	5.4	6.1	9.5	17.3	9.6	7.1	8.2	11.9	24.0	12.0
No. 50	2.9	3.5	4.6	5.9	3.3	4.6	5.2	6.9	7.8	4.9
No. 100					2.1					3.1
No. 200					1.3					1.9
Total Weight, g	250.0	251.0	251.0	251.0	1,003.0	249.8	249.8	250.0	250.0	999.5
(c) 100 Psi; 4% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	19.6	100.0			79.9	25.0	100.0			79.4
No. 3	6.2	25.4	100.0		57.9	11.0	29.6	100.0		58.9
No. 4	4.4	8.4	28.4	100.0	35.3	8.0	11.0	36.2	100.0	38.0
No. 6	3.0	4.8	11.6	49.4	17.2	5.0	7.0	16.2	55.2	20.2
No. 8	2.2	3.4	7.2	24.6	9.4	3.0	4.8	10.8	30.8	11.9
No. 16	1.1	1.5	3.5	10.5	4.1	1.8	3.0	5.7	14.9	5.7
No. 30	0.5	0.7	2.0	5.8	2.2	1.0	2.0	4.3	9.1	3.4
No. 50					1.3					2.1
No. 100					0.8					1.5
No. 200					0.5					1.0
Total Weight, g	250.0	250.0	250.0	250.0	1,000.0	250.0	250.0	250.0	250.0	1,000.0
(d) 200 Psi; 4% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	30.5	100.0			84.1	34.0	100.0			83.0
No. 3	14.9	36.5	100.0		62.9	17.0	43.0	100.0		64.5
No. 4	10.1	14.5	45.6	100.0	42.6	12.0	20.2	48.0	100.0	44.8
No. 6	7.9	10.3	25.4	60.6	28.1	8.6	13.6	29.2	65.4	29.2
No. 8	5.7	6.9	18.0	35.2	18.0	7.1	10.0	21.2	39.7	19.5
No. 16	2.9	3.8	9.2	20.2	9.1	4.1	5.8	12.6	23.5	10.5
No. 30	1.8	2.8	6.9	13.0	5.4	2.6	3.5	9.2	17.0	6.3
No. 50					3.4					3.9
No. 100					2.2					2.5
No. 200					1.5					1.6
Total Weight, g	250.0	250.0	250.0	250.0	1,000.0	250.0	250.0	250.0	250.0	1,000.0

TABLE 9

RESULTS OF SIEVE ANALYSIS OF COLORED AGGREGATES, GRADING B

Sieve	Total Percent Passing									
	30 Rev.					100 Rev.				
	½ to ⅜ In. Violet	⅜ In. to No. 3 Red	No. 3 to 4 Green	No. 4 to 6 Natural	Total	½ to ⅜ In. Violet	⅜ In. to No. 3 Red	No. 3 to 4 Green	No. 4 to 6 Natural	Total
(a) 100 Psi; 0% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	20.1	100.0			89.4	22.5	100.0			88.3
No. 3	6.0	19.0	100.0		67.4	6.8	19.6	100.0		67.5
No. 4	3.4	4.4	23.7	100.0	54.5	5.0	7.3	25.4	100.0	55.2
No. 6	2.5	3.1	9.1	90.2	48.8	3.2	5.7	12.5	93.2	49.7
No. 8	1.4	2.1	4.5	75.1	40.7	1.8	3.7	7.8	78.8	41.8
No. 16	0.4	0.6	1.3	40.5	20.5	0.7	2.4	4.3	43.4	22.2
No. 30	0.1	0.3	0.5	26.8	13.4	0.2	1.6	2.5	28.6	14.8
No. 50					7.8					8.7
No. 100					5.4					5.9
No. 200					3.6					4.0
Total Weight, g	140.0	240.0	120.0	499.0	999.0	140.0	240.0	120.0	498.0	998.0
(b) 200 Psi; 0% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	24.7	100.0			89.6	26.1	100.0			89.6
No. 3	9.2	26.7	100.0		69.9	10.5	30.0	100.0		70.2
No. 4	6.4	9.4	31.7	100.0	57.0	7.2	11.6	32.9	100.0	57.4
No. 6	4.4	6.8	14.2	95.2	51.1	5.8	7.4	17.1	96.7	51.8
No. 8	2.2	4.4	9.2	82.2	43.3	2.8	5.7	11.3	84.7	44.2
No. 16	1.1	3.2	5.5	45.0	23.3	1.7	4.2	6.7	47.9	34.4
No. 30	0.4	2.3	3.0	30.6	15.6	0.7	3.2	3.5	32.4	16.3
No. 50					9.2					9.8
No. 100					6.3					7.1
No. 200					4.2					4.8
Total Weight, g	140.0	240.0	120.0	499.0	999.0	140.0	240.0	120.0	500.0	1,000.0
(c) 100 Psi; 4% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	16.8	100.0			88.0	18.5	100.0			88.4
No. 3	3.9	23.7	100.0		65.8	4.7	24.6	100.0		68.6
No. 4	2.1	4.1	15.8	100.0	53.1	3.3	6.7	20.0	100.0	54.5
No. 6	1.7	2.4	5.4	93.5	48.1	2.2	4.6	9.2	93.7	49.4
No. 8	1.3	1.6	3.3	77.7	39.7	1.5	2.9	5.0	78.7	40.9
No. 16	0.7	1.0	1.6	40.3	20.4	0.9	1.5	2.1	41.1	21.2
No. 30	0.2	0.4	0.8	26.8	13.4	0.4	0.8	1.2	27.3	14.0
No. 50					9.0					9.4
No. 100					5.7					5.9
No. 200					3.6					3.8
Total Weight, g	140.0	240.0	120.0	500.0	1,000.0	140.0	240.0	120.0	500.0	1,000.0
(d) 200 Psi; 4% Asphalt										
½ In.	100.0				100.0	100.0				100.0
⅜ In.	19.7	100.0			89.0	21.4	100.0			89.9
No. 3	5.7	26.3	100.0		69.1	8.2	27.5	100.0		69.4
No. 4	4.0	8.6	27.1	100.0	56.4	6.0	10.7	35.5	100.0	56.8
No. 6	3.0	6.8	12.9	94.2	50.6	4.3	8.8	18.8	94.7	50.6
No. 8	2.4	3.7	7.5	80.2	42.3	3.2	6.0	13.8	81.7	43.2
No. 16	1.9	2.6	3.8	42.0	22.1	2.5	3.2	7.3	44.1	23.1
No. 30	1.0	1.9	2.6	28.5	15.0	1.5	2.3	5.5	29.3	15.4
No. 50					9.1					9.5
No. 100					6.4					6.6
No. 200					4.4					4.7
Total Weight, g	140.0	240.0	120.0	496.5	996.5	140.0	240.0	120.0	490.0	990.0

TABLE 10

RESULTS OF SIEVE ANALYSIS OF COLORED AGGREGATES, GRADING F

Sieve	Total Percent Passing									
	30 Rev.					100 Rev.				
	½ to ⅜ In. Violet	⅜ In. to No. 3 Red	No. 3 to 4 Green	No. 4 to 6 Natural	Total	½ to ⅜ In. Violet	⅜ In. to No. 3 Red	No. 3 to 4 Green	No. 4 to 6 Natural	Total
					(a) 100 Psi; 0% Asphalt					
½ In.	100.0				100.0	100.0				100.0
⅜ In.	15.7	100.0			86.7	18.9	100.0			87.7
No. 3	4.0	17.0	100.0		73.8	5.9	18.2	100.0		74.0
No. 4	2.6	4.7	16.8	100.0	63.9	3.9	5.7	21.1	100.0	64.2
No. 6	1.9	3.1	6.3	87.7	56.2	2.9	4.1	9.5	89.6	56.4
No. 8	1.2	2.2	3.7	74.1	47.4	1.8	2.8	6.3	76.5	47.8
No. 16	0.4	1.1	1.9	53.1	32.8	0.8	1.6	3.8	53.8	33.2
No. 30	0.1	0.7	1.1	37.5	23.0	0.5	0.9	2.3	38.5	23.6
No. 50					17.1					17.3
No. 100					13.3					14.3
No. 200					9.1					10.3
Total Weight, g	134.0	159.0	95.0	612.0	1,000.0	134.0	159.0	95.0	612.0	1,000.0
					(b) 200 Psi; 0% Asphalt					
½ In.	100.0				100.0	100.0				100.0
⅜ In.	21.7	100.0			89.5	32.1	100.0			90.9
No. 3	8.3	22.1	100.0		76.1	11.9	26.6	100.0		79.9
No. 4	4.9	9.8	25.3	100.0	65.8	8.5	12.3	38.9	100.0	68.4
No. 6	3.8	6.3	11.1	91.0	58.1	6.3	8.4	20.5	92.3	61.0
No. 8	2.4	4.8	7.8	79.0	49.7	4.1	5.5	12.1	81.7	52.9
No. 16	1.1	2.5	4.8	56.0	34.7	2.3	3.3	6.9	58.8	36.9
No. 30	0.8	1.7	3.0	40.5	25.0	1.4	2.5	4.3	43.5	26.9
No. 50					18.5					19.8
No. 100					15.6					16.3
No. 200					11.6					12.9
Total Weight, g	134.0	159.0	95.0	610.0	998.0	134.0	159.0	95.0	616.0	1,000.0
					(c) 100 Psi; 4% Asphalt					
½ In.	100.0				100.0	100.0				100.0
⅜ In.	11.2	100.0			88.0	15.4	100.0			89.6
No. 3	5.2	14.5	100.0		73.5	6.0	16.0	100.0		74.4
No. 4	3.7	7.3	25.3	100.0	64.1	4.1	8.2	28.9	100.0	64.4
No. 6	2.0	5.7	12.7	83.5	53.4	2.6	6.1	14.1	85.5	54.3
No. 8	1.9	4.0	9.0	73.2	46.5	2.1	4.4	11.9	75.4	47.7
No. 16	0.7	1.6	5.0	52.6	32.4	0.9	2.0	6.0	53.5	33.3
No. 30	0.2	0.6	1.2	38.0	23.1	0.4	1.0	2.0	38.9	24.0
No. 50					17.0					17.6
No. 100					12.4					12.8
No. 200					8.7					8.9
Total Weight, g	134.0	159.0	95.0	607.0	995.0	134.0	159.0	95.0	612.0	1,000.0
					(d) 200 Psi; 4% Asphalt					
½ In.	100.0				100.0	100.0				100.0
⅜ In.	18.3	100.0			89.9	26.5	100.0			90.1
No. 3	6.6	16.9	100.0		74.6	7.7	17.6	100.0		74.9
No. 4	4.8	8.8	36.3	100.0	65.4	5.4	9.6	47.9	100.0	65.9
No. 6	2.9	6.8	15.8	87.8	56.3	3.5	7.4	16.4	88.9	56.5
No. 8	2.4	5.0	12.6	77.9	49.6	3.0	5.8	13.1	78.2	50.5
No. 16	1.2	2.3	7.0	55.5	34.5	1.6	3.2	8.1	56.4	35.2
No. 30	0.7	1.2	2.9	40.5	24.8	1.1	1.8	3.6	42.0	25.4
No. 50					18.4					19.0
No. 100					13.4					14.1
No. 200					9.1					10.3
Total Weight, g	134.0	154.0	95.0	612.0	995.0	134.0	159.0	95.0	605.0	993.0

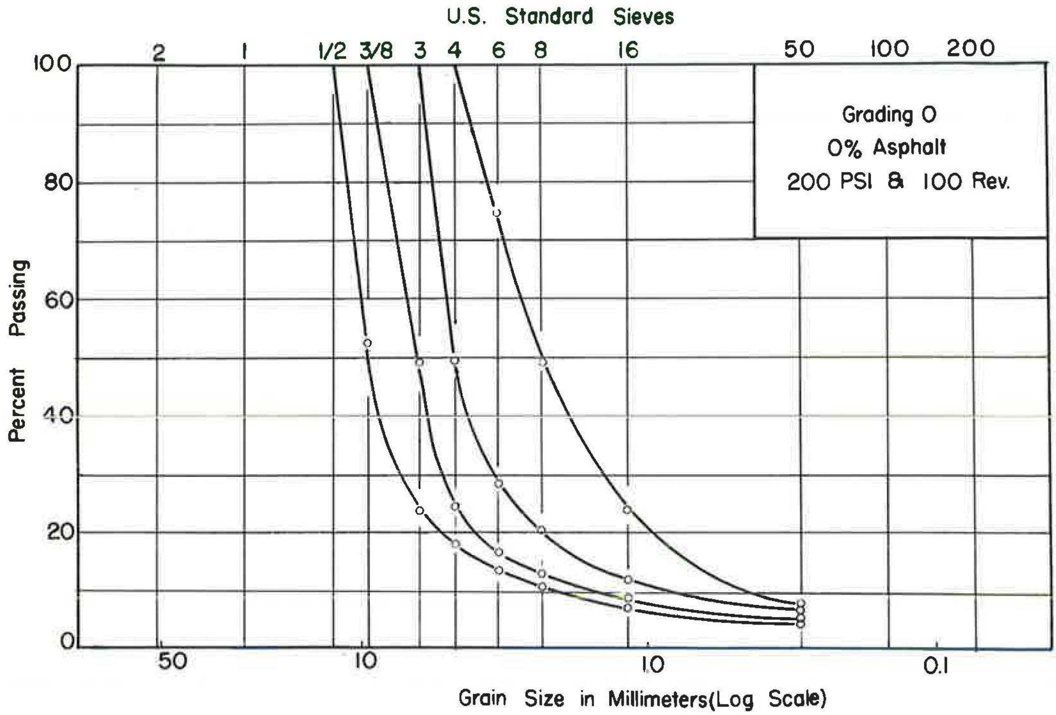

Figure 9. Sieve analysis of compacted colored aggregate.

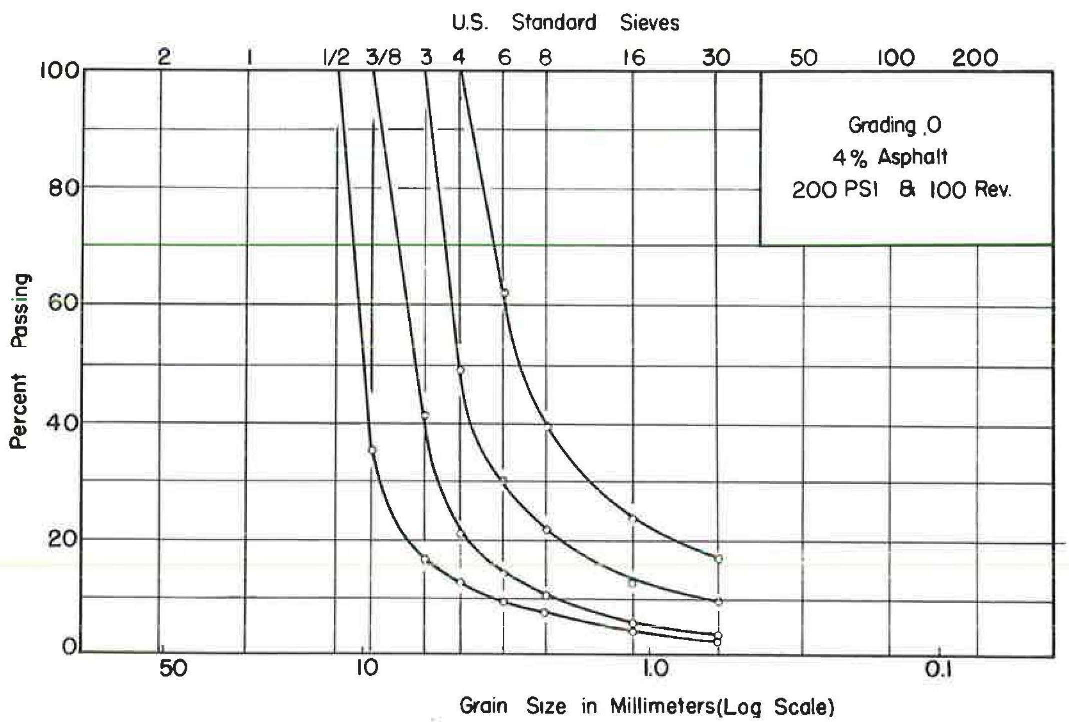

Figure 10. Degradation vs number of revolutions for one-sized aggregates.

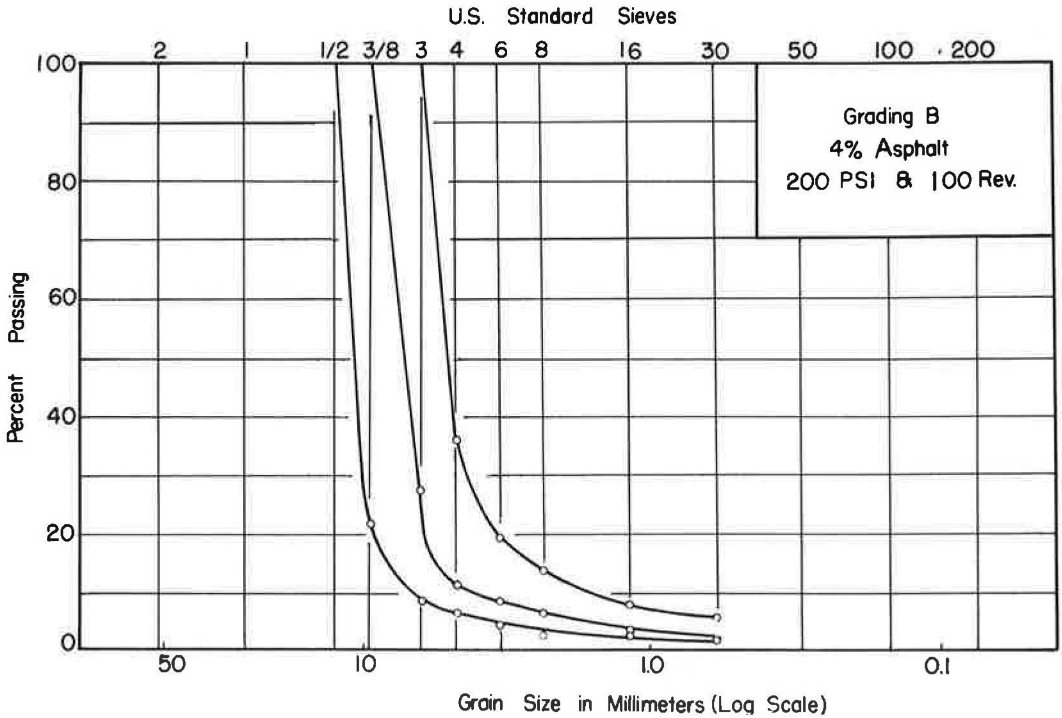

Figure 11. Degradation vs number of revolutions for one-sized aggregates.

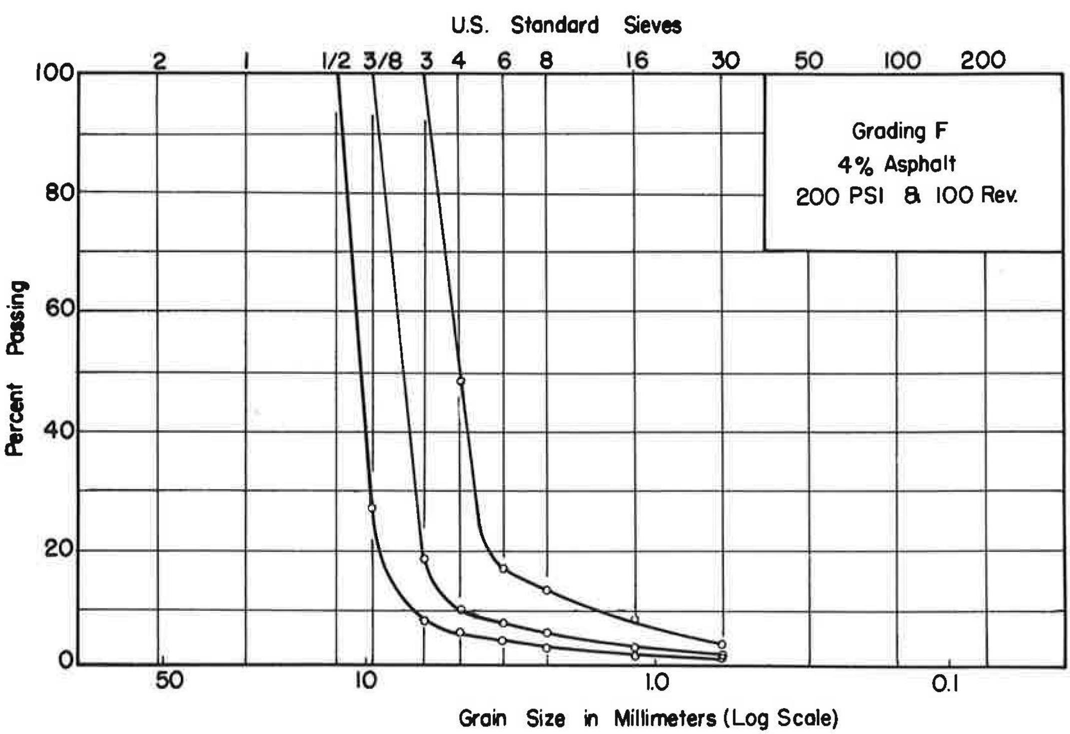

Figure 12. Degradation vs number of revolutions for one-sized aggregates.

each fraction is constant, then the combination of particles of different sizes will have a pattern which depends only on the blending ratios of these sizes rather than on type of aggregate or magnitude of compactive effort.

Thus, it can be stated that if pattern of degradation is a matter of concern, which is the case in ore treatment and in mining and metallurgical engineering, then this pattern can be predicted beforehand by knowing the gradation of feed material. But if magnitude of degradation is a matter of concern, additional variables have to be investigated thoroughly before any prediction can be made concerning this factor. In other words, in addition to gradation, the magnitude of degradation in a degradation process is dependent upon compactive effort, shape of particles, and type of rock even though these factors do not affect its pattern. For example, a change of gradation will not eliminate production of a certain size of particles when particles of larger size than this size are produced. The change in gradation will reduce or increase each size in such a proportion that the final gradation of each fraction will follow a smooth curve approaching a parabolic one. However, this change of gradation will change the magnitude of degradation, because the magnitude of degradation depends on energy consumed for breakage. So any factor affecting the breakage energy will affect the magnitude of degradation. For example, higher compactive effort corresponds to higher breakage energy and thus has to result in higher degradation. But the pattern of degradation is not energy dependent and can be considered as a constant.

Since, for any original gradation, the pattern of degradation is constant, and it is only the magnitude of degradation which varies with other factors, it can be deduced that the effects of degradation on the properties of a given bituminous mixture have to be due to the magnitude of degradation. Therefore, in the detailed study which follows, only the magnitude of degradation has been considered, and attempts are made to find which factors are more effective in reducing the magnitude of degradation and what protective measures can be taken against degradation of aggregate in bituminous mixtures.

Effect of Mixture and Compaction Variables

The magnitude of degradation, measured by percent increase in surface area, was determined for the three types of aggregate, dolomite, limestone, and quartzite. Three gradations, grading O, grading B, and grading F, were used. Compactive effort applied by the gyratory compactor was changed both in ram pressure and number of revolutions. For this purpose 450 specimens were formed and tested, the asphalt was extracted, and a sieve analysis made on the dry aggregate from which the percent increase in surface area for each specimen was calculated.

Table 11 gives data for the percent increase in surface area for each of the three kinds of aggregate.

Ram Pressure and Number of Revolutions.—Figure 13 shows the percent increase in surface area versus number of revolutions for specimens made of limestone with 0 and 4 percent asphalt. All specimens were made of grading O. The ram pressures are indicated on each curve. Degradation increases very rapidly in the first part of the test and then continues to increase at a decreasing rate until about 250 revolutions after which the rate of increase remains constant in each case. Also, as ram pressure increases the degradation in the first few revolutions increases drastically. For a ram pressure of 250 psi, almost 70 percent of the degradation that occurred at 1,000 revolutions had occurred in the first hundred revolutions, whereas at 50-psi ram pressure only 50 percent of the degradation had occurred in the first hundred revolutions.

Figures 14 and 15 show degradation versus ram pressure for specimens made of limestone with 0 and 4 percent asphalt; the results for all three gradings are shown. Degradation on the ordinate is plotted on a log scale; ram pressure on the abscissa is plotted to an arithmetic scale. Gradation designations of original mixtures are shown at the left side of the curves. Degradation increases both with increase in ram pressure and increase in number of revolutions; therefore, degradation increases with increase in compactive effort.

Figures 16 and 17 plot degradation versus number of revolutions. Each curve is for a single ram pressure, as indicated. Degradation for each gradation is plotted on a

TABLE 11

PERCENT INCREASE IN SURFACE AREA

Psi	Rev.	Grading O, % Asphalt				Grading B, % Asphalt				Grading F, % Asphalt			
		0	2	4	6	0	2	4	6	0	2	4	6
		(a) Dolomite											
50	30	258.0				24.0				11.2			
	100	321.0				35.5				16.3			
	250	420.0				44.0				20.0			
	500	500.0				72.2				23.9			
100	30	334.2	309.0	308.0	395.0	41.7	39.7	40.0	47.2	14.4	15.1	12.4	13.1
	60	422.0	382.0	370.0	408.0	52.3	44.5	46.8	51.5	21.0	16.5	14.2	16.3
	100	500.0	410.0	416.0	419.0	61.0	49.5	53.0	59.4	24.5	17.5	17.0	18.9
	250	660.0	470.0	485.0		74.0		60.5		32.8		22.0	
	500	740.0		600.0		105.0		65.5		37.0		25.5	
200	30	628.0	571.0	594.0	563.0	62.3	52.0	62.3	63.4	25.4	19.0	17.5	24.3
	60	805.0	655.0	680.0	734.0	77.1	61.0	68.2	68.3	30.0	22.2	22.7	28.8
	100	937.0	706.0	752.0	757.0	90.0	66.7	75.0	72.0	32.3	25.5	26.5	30.7
	250	1,250.0	890.0	915.0		120.0		84.5		39.0		33.0	
	500	1,440.0		1,070.0		146.0		92.0		44.0		37.0	
250	30	730.0	646.0	648.0	698.0		60.5	69.6	70.6		21.1	20.0	25.3
	60	881.0	775.0	780.0	840.0		70.3	79.3	76.5		25.2	23.9	31.7
	100	1,058.7	859.0	892.0	919.0		80.0	82.0	78.2		29.0	28.6	38.1
	250	1,480.0	1,000.0	1,050.0				95.0				36.2	
	500	1,700.0		1,230.0				102.0				41.5	
		(b) Limestone											
50	30	85.0		68.4		19.6				5.2			
	60	120.5		105.3									
	100	175.5		134.0		30.5				7.4			
	250	220.0		158.0									
	500	275.0		185.0		45.1				14.1			
	1,000	378.0		249.0									
100	30	238.0	204.0	180.0		31.1	25.6	39.7	37.9	11.0	10.5	10.2	11.2
	60	278.0	275.0	255.0		40.6	31.9	45.5	40.3	15.4	14.2	11.2	15.0
	100	320.0	310.0	290.0		47.0	35.0	49.0	42.1	16.9	16.0	13.3	17.5
	250	390.0	365.0	355.0		58.5		54.5		21.6		16.8	
	500	462.0		390.0		72.0		64.0		25.6		18.4	
	1,000	580.0		484.0									
200	30	430.0	374.0	380.0		51.5	43.1	54.8	52.7	15.3	17.0	17.8	17.5
	60	510.0	440.0	493.0		57.9	47.5	60.6	57.3	20.5	21.0	20.0	20.5
	100	594.0	510.0	552.0		64.1	52.5	64.0	64.0	24.5	24.0	22.5	25.5
	250	678.0	600.0	625.0		72.0		76.0		30.0		26.5	
	500	765.0		681.0		90.0		83.6		32.5		30.0	
	1,000	929.0		776.0									
250	30	526.3	427.0	502.0			46.7	59.5	54.0		18.1	19.3	20.6
	60	588.6	559.0	570.0			50.0	66.0	62.1		23.0	22.2	25.8
	100	678.9	639.0	630.0			55.0	71.6	66.0		28.5	26.2	31.2
	250	779.0	720.0	726.3				80.0				30.7	
	500	900.0		807.9				88.0				32.8	
	1,000			955.3									
		(c) Quartzite											
50	30					11.2				2.0			
	100					18.1				4.8			
	250					25.0				7.9			
	500					28.8				13.5			
100	30	126.0	154.0	149.0		15.0	12.6	15.7	21.4	4.3	2.3	3.6	4.3
	60	179.0	202.0	164.0		20.0	20.7	21.5	23.9	7.0	5.3	5.2	6.5
	100	196.0	236.0	198.0		24.9	22.8	30.0	25.9	8.6	8.8	7.0	7.9
	250	230.0	284.0	229.0		33.9		37.5		15.0		9.5	
	500	300.0		270.0		39.0		44.0		18.0		12.5	
200	30	261.0	245.0	250.0		28.4	26.2	27.5	39.1	7.5	7.0	7.0	8.6
	60	334.0	280.0	300.0		37.0	35.6	34.6	42.5	10.3	8.9	9.3	12.1
	100	364.0	338.0	335.0		43.4	37.9	41.2	49.2	13.5	12.0	12.1	15.8
	250	440.0	400.0	405.0		53.8		49.0		18.9		15.5	
	500	530.0		460.0		61.8		58.0		23.8		18.6	
250	30	292.0	300.0	300.0			34.1	32.5	45.0		11.4	9.3	10.2
	60	380.0	325.0	352.0			38.0	38.4	49.6		12.4	11.3	13.6
	100	420.0	370.0	420.0			42.8	45.0	54.5		14.5	15.0	17.0
	250	511.0	444.0	500.0				52.0				17.5	
	500	610.0		560.0				60.0				21.1	

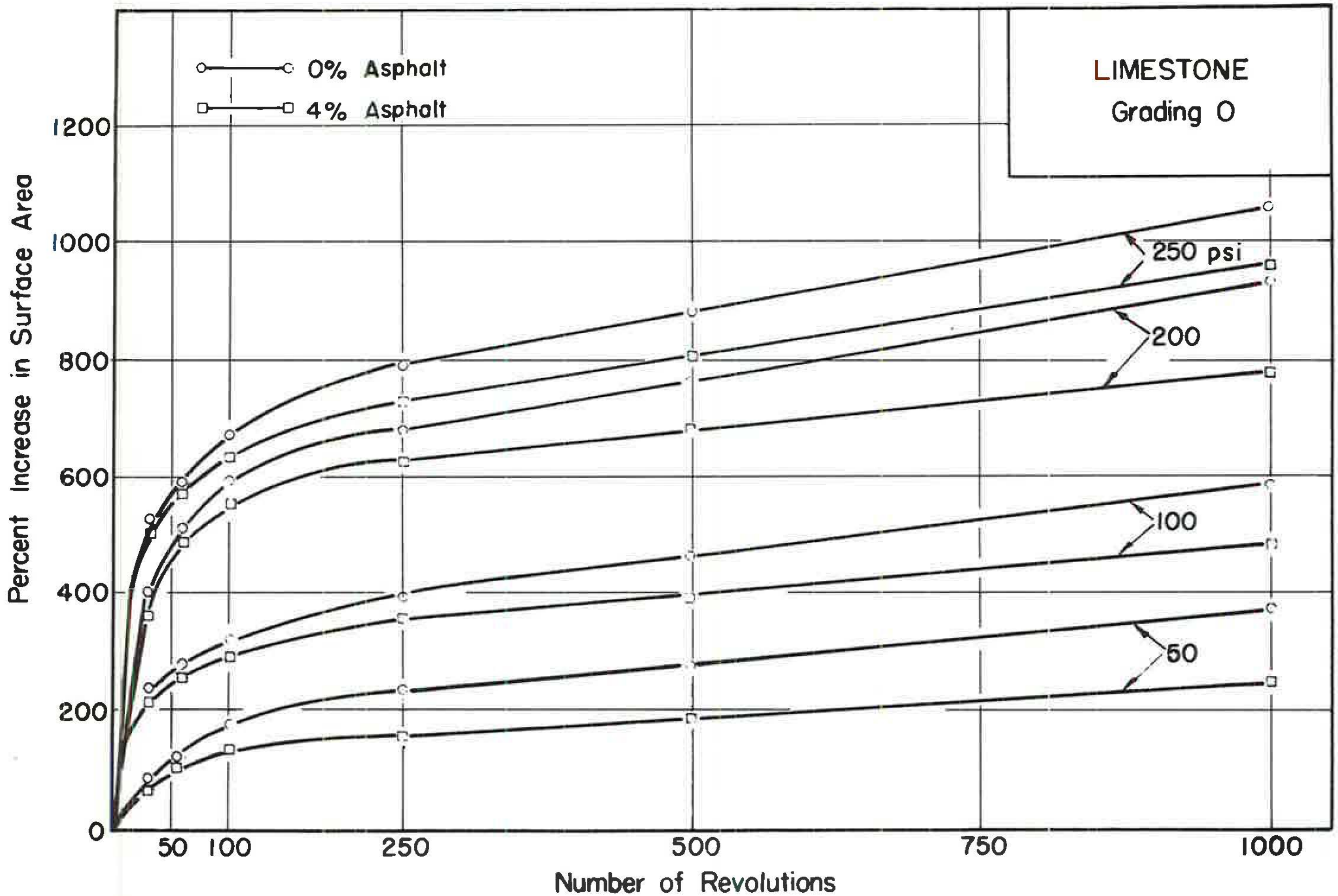

Figure 13. Degradation vs number of revolutions; variable ram pressure.

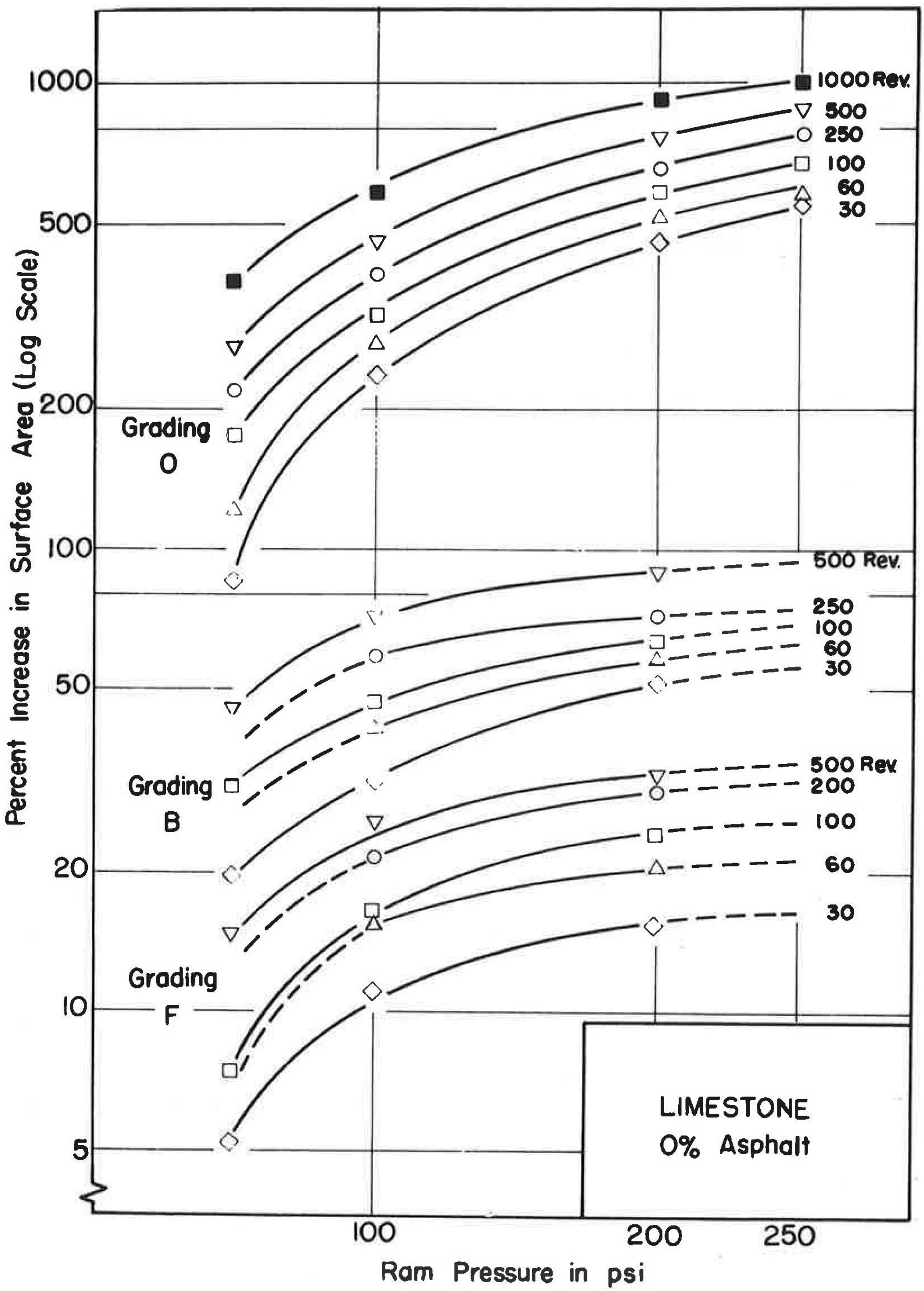

Figure 14. Degradation vs ram pressure.

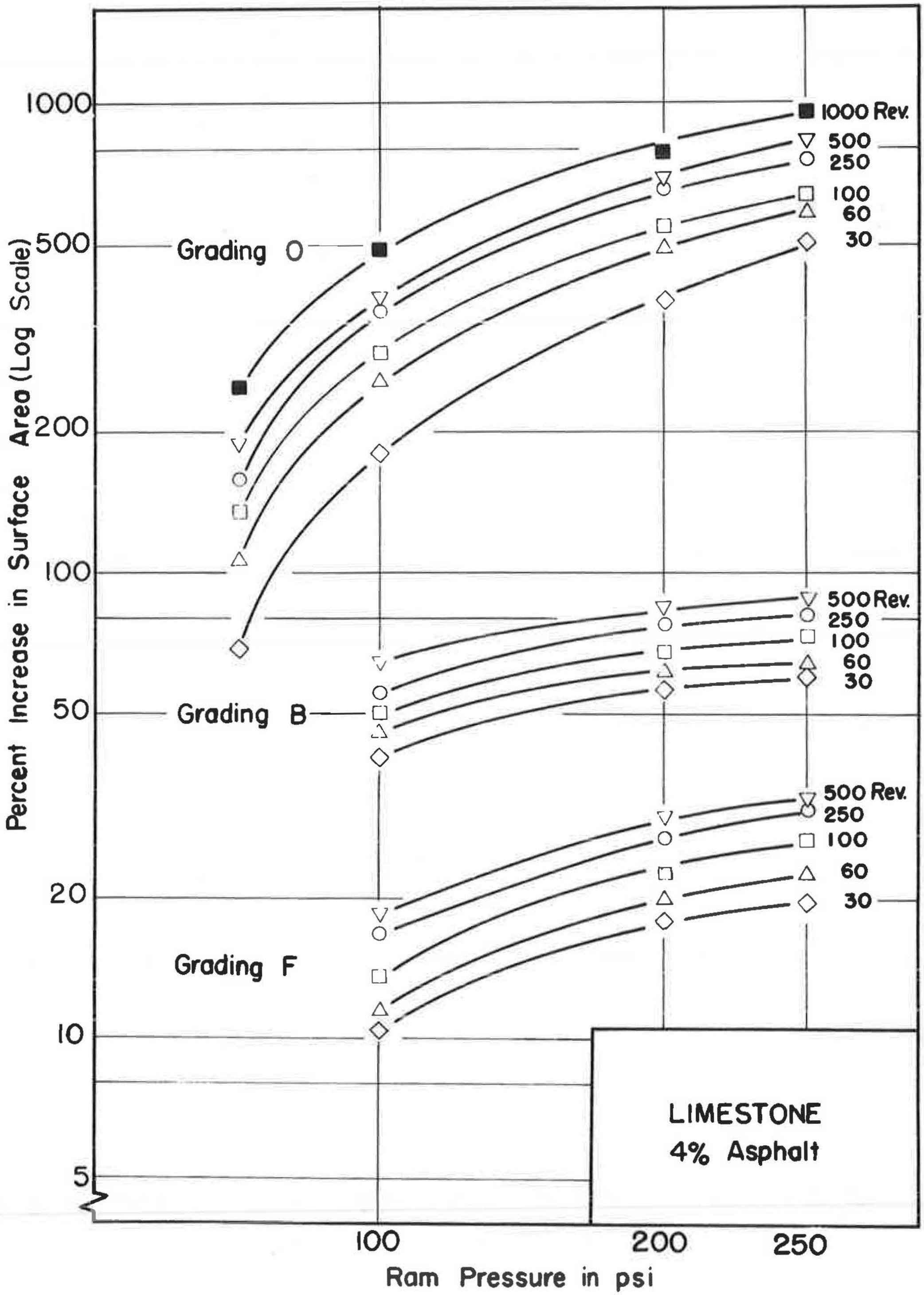

Figure 15. Degradation vs ram pressure.

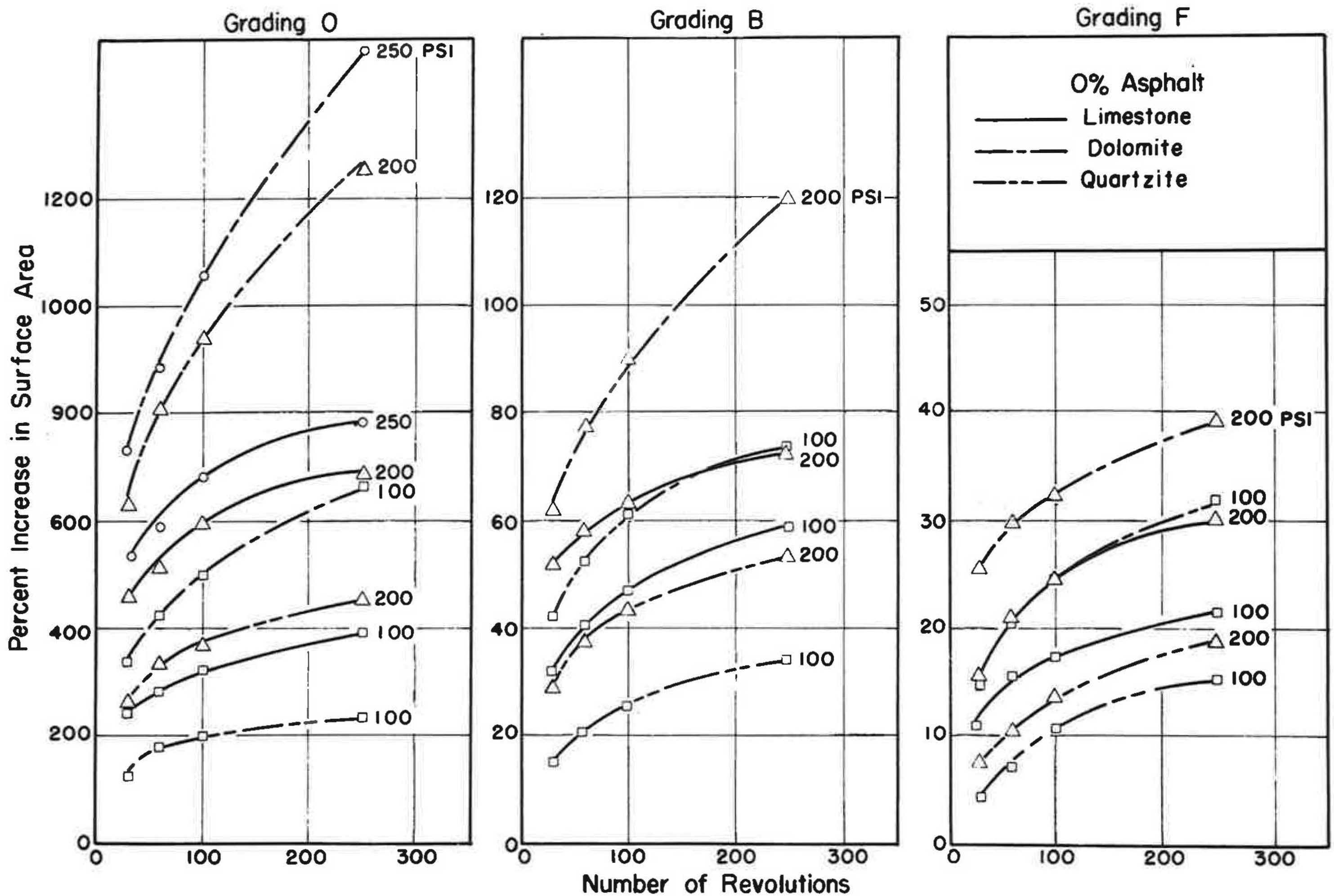

Figure 16. Degradation vs number of revolutions.

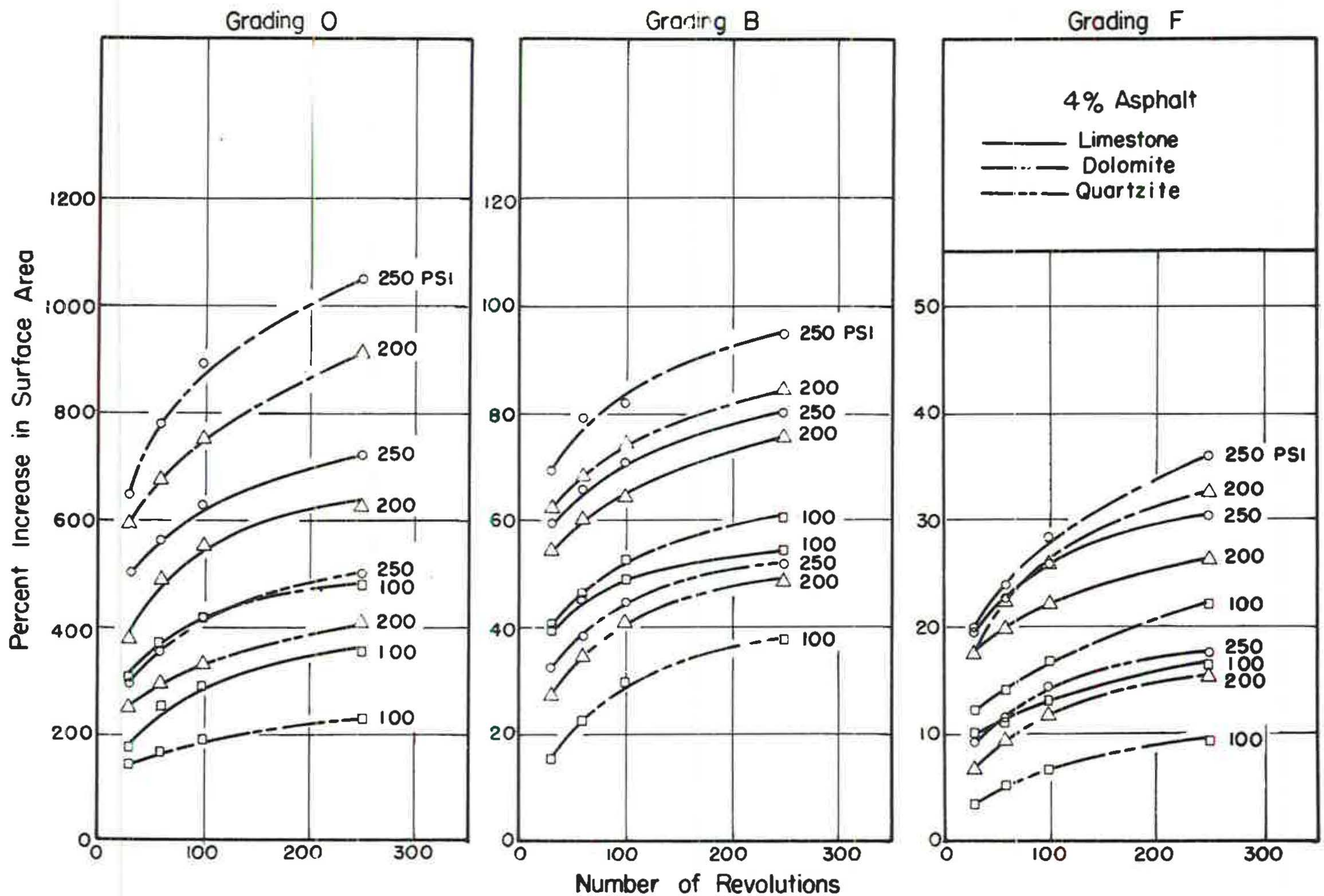

Figure 17. Degradation vs number of revolutions.

different scale. These figures also indicate that as compactive effort increases degradation also increases.

When ram pressure was kept constant and compactive effort was increased only by the number of revolutions, the increase in degradation depended on type of aggregate and gradation of aggregate. The softer and weaker the aggregate (higher Los Angeles value) the greater was the increase in degradation caused by increase in number of revolutions, while the harder (lower Los Angeles value) the aggregate the less was the increase in degradation. Figures 16 and 17 also show that increase in degradation caused by increase in number of revolutions depends on gradation. The slopes of curves for open-graded mixtures are much steeper than those for dense-graded ones.

Type and Gradation of Aggregate. —Even more pronounced than the effect of compactive effort is the effect of the original gradation of the mixture on the degradation of aggregate. As gradation becomes more dense, degradation decreases (Figs. 14 and 15). Open-graded mixtures which contain only the four top sizes of aggregate produced the highest degradation for all three kinds of aggregate, at all compactive levels, and for all asphalt contents. At the same time, grading F which corresponds to Fuller's gradation for maximum density gave the lowest values of degradation under the same conditions. Although it is not at once apparent because a log scale has been used to plot degradation, it should be noted that open-graded mixtures experienced some twenty times more degradation than dense-graded mixtures under the same conditions.

Figures 16 and 17 indicate that the amount of degradation also depends on kind of aggregate. The softer and weaker (higher Los Angeles value) the aggregate the more the degradation. The curves for dolomite always lie above the curves for the other two kinds of aggregate. However, the effect of aggregate softness and strength on degradation also depends on gradation of the mixtures. For example (Fig. 16), the change in degradation due to kind of aggregate is a matter of a few hundred percent for the case of the open-graded mixtures, while for the dense-graded mixtures this change is around 50 percent at most.

Cognizance of the scale of degradation for each gradation makes one aware that original gradation of aggregate has a very pronounced effect on magnitude of degradation. Degradation for open-graded mixtures (grading O) ranges from 100 percent to 1,400 percent depending on the type of aggregate and compactive effort; while for dense-graded mixtures (grading F) this range is between 5 and 40 percent, or only about $1/20$ to $1/35$ of the values obtained for open-graded mixtures. This indicates that the original aggregate gradation is the most important factor in degradation, because the results indicate that changes in compactive effort, in kind of aggregate, or in aggregate shape (as discussed later) did not produce as much change in degradation as changes in original gradation.

This point can easily be related to the previous finding with regard to mechanism of degradation. It was said that magnitude of degradation depends on distribution and magnitude of forces applied to the specimen. When a dense mixture is used the number of contact points is numerous and any applied force will be distributed to many more points in much less intensity than for more open mixtures, which in turn produces much less breakage. In open mixtures the number of contact points are few, and particles are subjected to much higher contact pressures, which in turn causes much more breakage than in dense-graded mixtures.

Asphalt Cement. —Figure 18 shows the effect of change in asphalt content on degradation for the three gradings of limestone aggregate. Depending on compactive effort, kind of aggregate, and gradation of aggregate there is in general an asphalt content for which the degradation is minimum. It is also indicated that asphalt content is not an independent variable with respect to degradation as was shown to be the case for kind of aggregate and aggregate gradation. For an independent factor, such as kind of aggregate, it could be said that when aggregates become softer and weaker the degradation increases regardless of other variables, but for the asphalt content variable there is no such trend.

This result may be viewed with respect to the role of asphalt in the mechanism of degradation. It was found that magnitude of degradation depends on distribution of load and intensity of contact pressure. Considering asphalt as a viscous material which

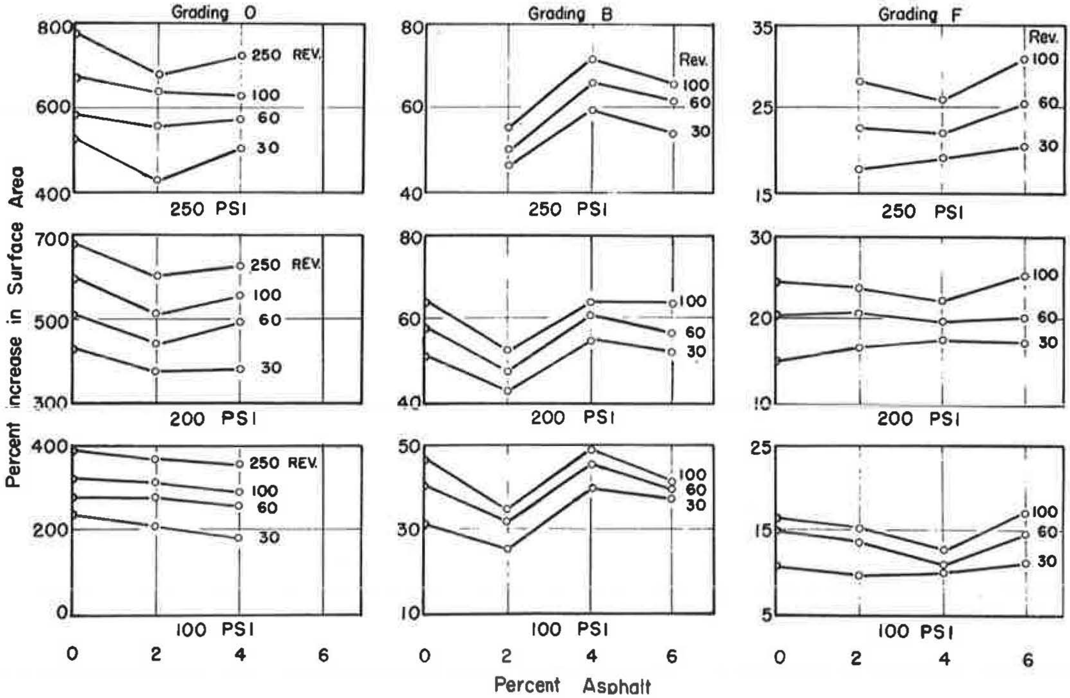

Figure 18. Degradation vs asphalt content, limestone.

covers the particles, its effect on degradation may be influenced by the effect of its viscosity on magnitude of contact pressure. Also, for a particular arrangement of particles and a particular condition of load the asphalt may help the particles to rotate and slip over each other. Rotation and slippage of particles will increase the probability of wear of corners of particles and will also increase the probability of obtaining a denser mixture. If these effects result in an increase in contact pressure, degradation will increase, but if the effect is to reduce contact pressure, degradation will be decreased. Since these effects of asphalt change as the specimen undergoes densification, the net result is a complex one in which no definite pattern for effect of asphalt on degradation is apparent.

Aggregate Shape. —To investigate the effect of aggregate shape on degradation, a limited number of tests were performed on specimens made of rounded pieces of quartzite. Table 12 contains the percent increase in surface area for such specimens. The same gradings (O, B, and F) were used in this part of the study. Eighteen specimens of each grading were tested, nine without asphalt and nine with 4 percent asphalt; therefore, a total of 54 specimens were used.

Figure 19 shows the results obtained from specimens with 4 percent asphalt, comparing rounded and angular quartzite. Curves for rounded aggregate lie below those for the angular material. Also, both the flatness and spacing of the curves for rounded pieces are less than those for angular ones, indicating that increase in compactive effort produces less degradation in the case of rounded aggregate regardless of whether the increase is due to pressure or number of revolutions. The cause of this phenomenon can be attributed to the reduction, in the case of rounded aggregate, of that part of degradation which is due to wear rather than breakage. Wear occurs due to the rounding off of corners of particles when they rotate or slip over each other. Breakage occurs when the contact pressure between two particles exceeds their strength, resulting in fracture or splitting. Theoretically, by using rounded particles that portion of degradation due to wear should be eliminated. Practically, however, this portion can only be reduced rather than eliminated, because when particles start to break, the newly produced pieces are no longer rounded and wear begins.

TABLE 12

PERCENT INCREASE IN SURFACE AREA, ROUNDED QUARTZITE

Psi	Rev.	Grading O, % Asphalt		Grading B, % Asphalt		Grading F, % Asphalt	
		0	4	0	4	0	4
100	30	67.8	82.9	7.2	10.8	1.0	0.7
	100	116.0	110.0	14.0	16.5	1.9	3.2
	250	138.0	135.0	19.0	20.5	4.2	6.0
200	30	114.0	142.4	12.2	20.0	2.6	2.5
	100	178.0	173.4	21.5	23.5	4.8	5.5
	250	212.0	198.0	28.0	28.5	7.7	8.0
250	30	128.0	175.0	13.3	23.3	2.9	4.5
	100	185.0	215.0	23.0	27.5	5.7	6.2
	250	231.0	250.0	29.0	32.0	8.6	9.0

This reasoning leads to the conclusion that the major part of the difference between degradation of rounded and angular particles can be considered as reduction of wear. Figure 19 shows that the rounded aggregate experienced almost 50 percent less degradation than the angular one, which then can be considered as almost 50 percent less wear. This reduction of degradation due to the shape of particles should decrease as softer

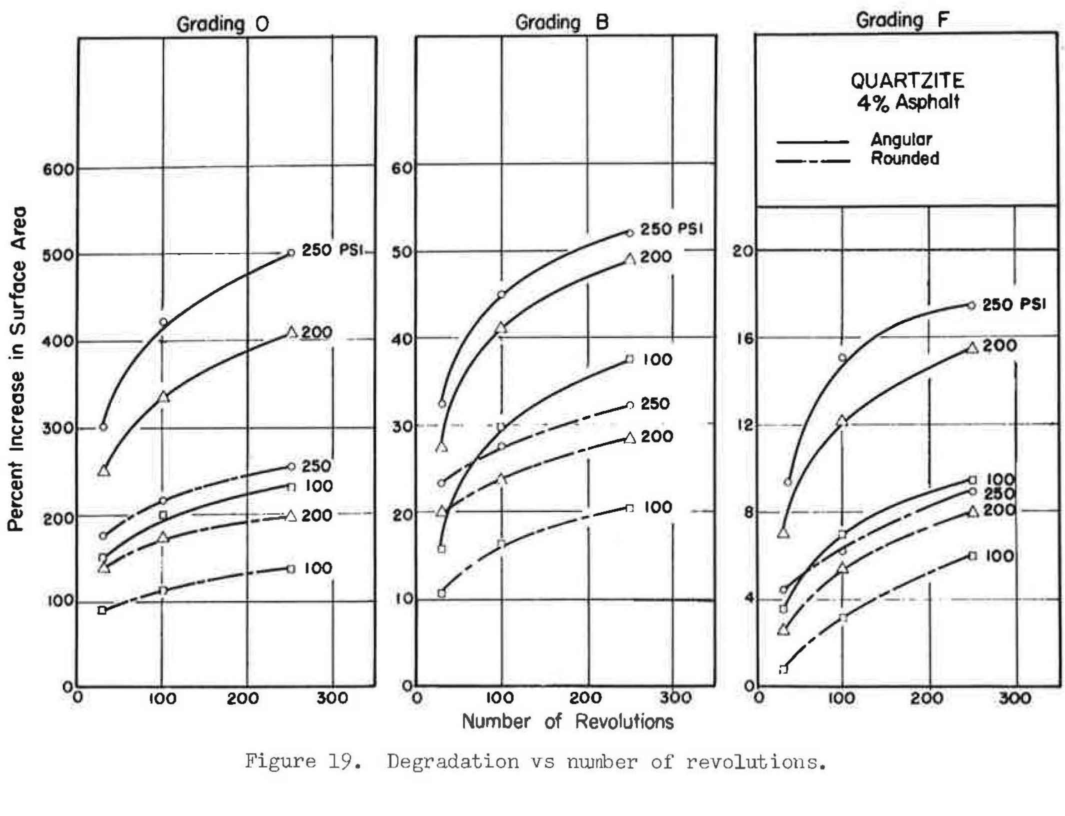

Figure 19. Degradation vs number of revolutions.

material is used, because in soft aggregates probability of breakage is high. Thus, after a few applications of load, the amount of angular pieces should increase and wear should start. This was one reason for using the quartzite with the lowest Los Angeles value in this portion of the study.

Degradation vs Los Angeles Value

Degradation values were plotted against the Los Angeles values for the three kinds of aggregate to determine any relationship. Grading C was used to determine the correlation between Los Angeles value and degradation merely because the maximum size of grading C is the closest to the maximum size used in this investigation.

Figures 20, 21, and 22 show the results obtained from gradings O, B, and F, respectively. Each curve is for the indicated number of revolutions. The three points on each curve are the results obtained from specimens made of the three kinds of aggregate tested under equal efforts.

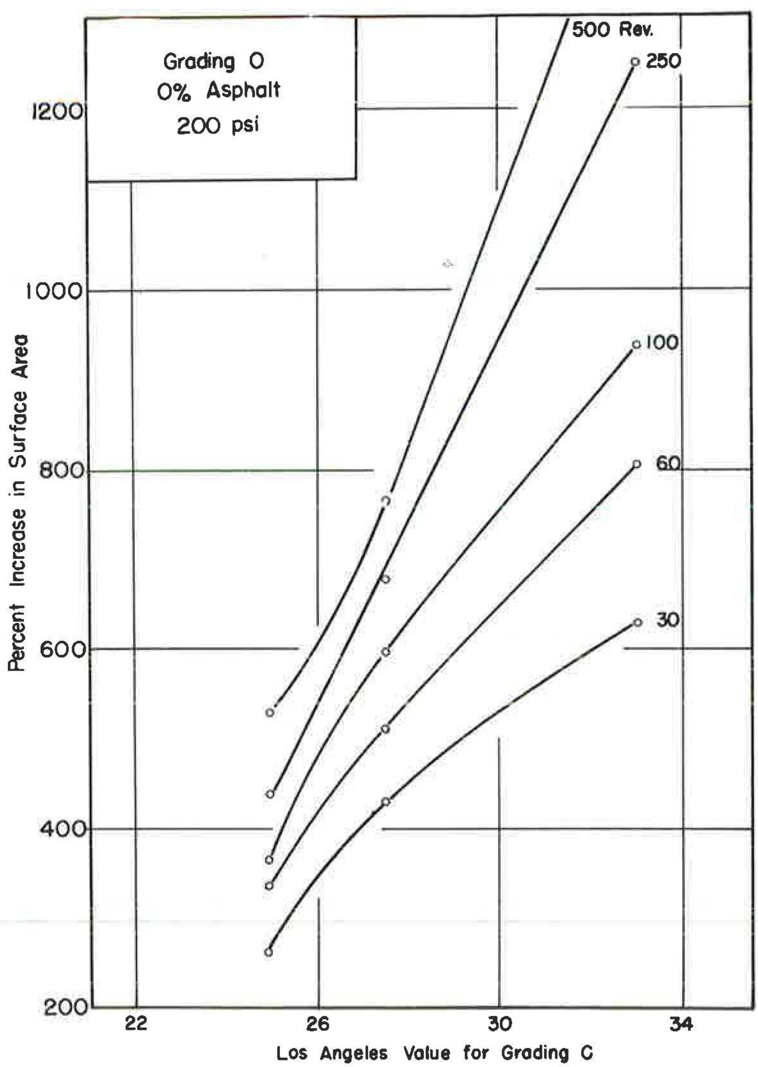

Figure 20. Degradation vs Los Angeles value.

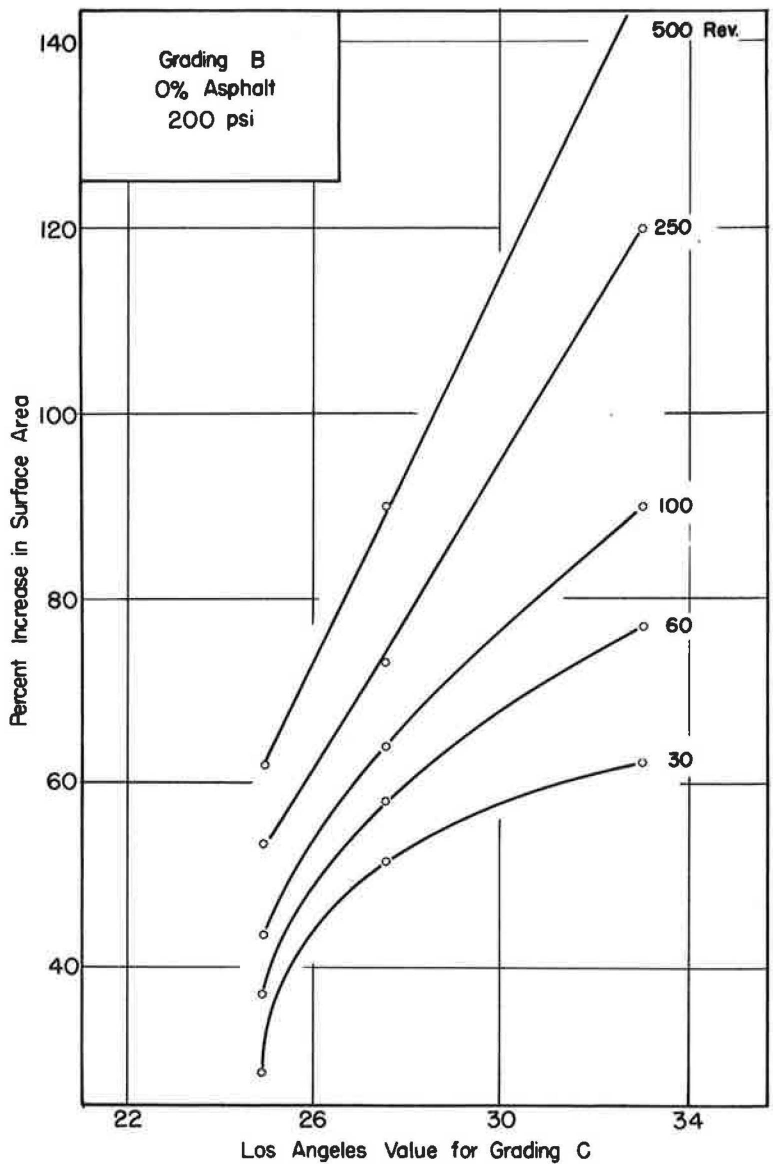

Figure 21. Degradation vs Los Angeles value.

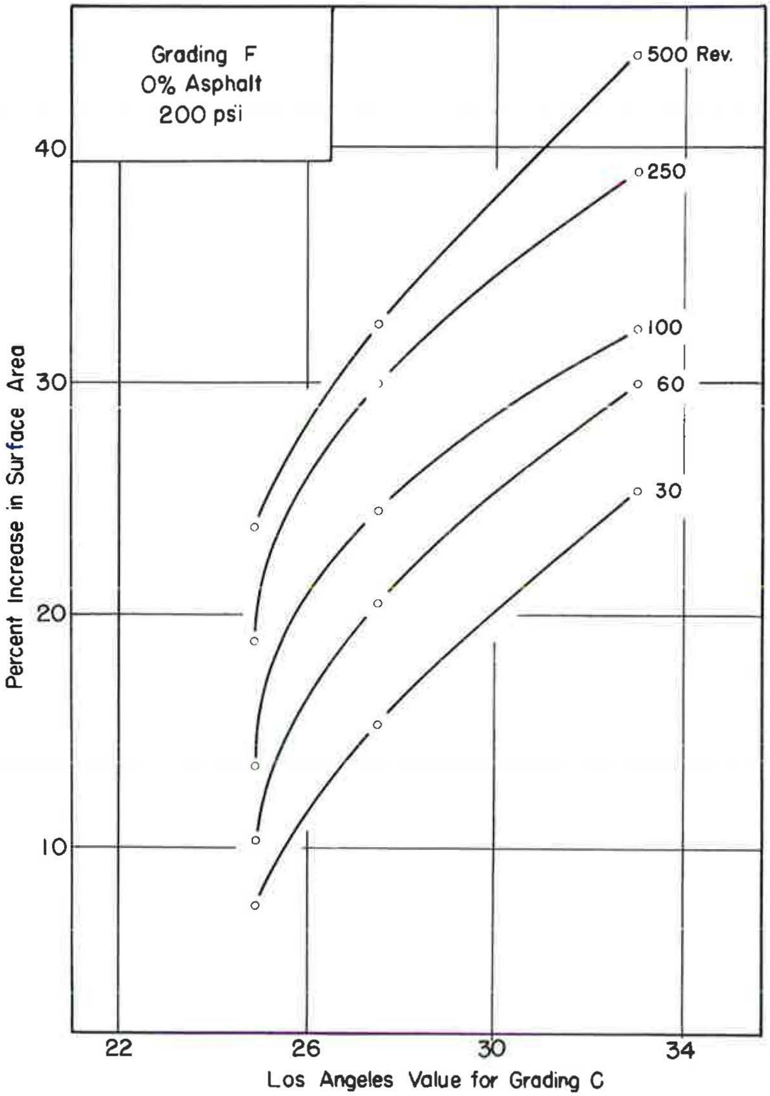

Figure 22. Degradation vs Los Angeles value.

Figure 20 shows that as the Los Angeles value increases the degradation value also increases, but the rate of increase is not constant, and the relationships are not linear until the compactive effort is about 200-psi ram pressure and 250 revolutions. Below this level of compactive effort the Los Angeles machine produces more degradation for soft or weak aggregate than the gyratory machine. Above 250 revolutions more degradation is experienced by the less resistant material in the gyratory compactor than in the Los Angeles machine because the curve for 500 revolutions is concave rather than convex.

Figure 21 shows that for grading B this linearity occurs somewhere between 200-psi ram pressure and 250 revolutions, and 200-psi ram pressure and 500 revolutions; Figure 22 shows that such linearity was not reached for specimens with grading F under compactive efforts used in this study.

It is therefore indicated that, depending on gradation of the aggregate, there is a certain level of compaction for which the plot of degradation versus Los Angeles value of the aggregate is a straight line. For compactive efforts higher than that, soft and weak aggregates experienced more degradation in the gyratory machine than in the Los Angeles machine, and for compactive efforts below that, soft and weak materials experienced more degradation in the Los Angeles machine. Therefore, as far as degradation is concerned, depending on the gradation of the material, the Los Angeles test corresponds only to a certain level of compaction. This level of compaction increases as gradation of material becomes more dense. Inasmuch as these levels of compaction, especially in dense-graded materials, are much higher than those normally found in the field, some doubts are imposed on the validity of the Los Angeles test as a measure of quality of aggregate with respect to degradation. This becomes especially apparent when it is noted that the dolomite aggregate with a high Los Angeles value (Figs. 16 and 17) when tested in a Fuller gradation produced less than one-tenth of the degradation under equal compactive effort of that produced by the low Los Angeles value quartzite when tested in the open gradation.

It was mentioned before that degradation occurs due to two phenomena, wear and breakage. Wear was considered responsible for that portion of degradation which is caused by rotation and slippage of particles over each other; breakage was considered to occur when the contact pressure exceeds the strength of the particle in a certain direction. Thus, under traffic compaction the particles either break or rotation wears off their corners. In either case the result is production of particles of smaller sizes. Rotation and breakage will result in a denser packing, producing a mat whose particles have more contact points and less chance for rotation. This reduces the rate of degradation under further compaction. But in the Los Angeles rattler test the particles do not experience this dense packing or cushioning effect which occurs in a road mat and consequently the material is subjected to a more severe degradation condition than actually exists in the field.

Petrographic Analysis

A comparison of petrographic analysis (Table 2) with degradation and Los Angeles values of the materials indicates that the nature of grain boundaries, cementation, and percent of voids influence the resistance of aggregates to degradation. Good interlocking between the grains in limestone results in a low Los Angeles value and low degradation. Loose interlocking in dolomite results in a high Los Angeles value and high degradation. Quartzite's strength is due to silica cementation that results in a comparatively strong and resistant rock. If the material had not been highly stressed, this strong cementation would have resulted in a very low Los Angeles value. However, the directional weakness due to cracking and fracturing makes the material susceptible to impact breakage, which may be the reason for its high Los Angeles value as compared to the nature of its cementation. The results also show that degradation increases as percent voids of the material increases.

CONCLUSIONS

The results obtained from this study led to the following conclusions, which are specifically applicable only to the particular kinds of aggregate used. Furthermore, all the tests were performed in the laboratory, and there exists no field correlation study specifically to evaluate the field behavior of the materials. All conclusions and recommendations deal with degradation characteristics of mineral aggregate. Suggested protective measures are made only with respect to the reduction of aggregate degradation without considering their effects on other properties of mixtures.

1. Within the range of the materials and procedures, there appears to be a unique pattern for degradation of each aggregate fraction of a bituminous mixture. This pattern does not vary with kind of aggregate, compactive effort, presence of asphalt, or original gradation of the mixture.
2. The magnitude of degradation of a bituminous mixture, as measured by percent increase in aggregate surface area, depends on the following factors: kind of aggregate, aggregate gradation, compactive effort, and particle shape. The effect of asphalt on the magnitude of degradation depends on other factors and cannot be considered as an independent variable.

3. Physical characteristics of the aggregate, as reflected by its Los Angeles value or by petrographic analysis, have a dominant effect on degradation. Mineral aggregates with low Los Angeles values will produce less degradation than those with high Los Angeles values. Rocks with good interlocking or cementation between grains are more resistant to degradation than others.

4. From the results of tests on mixtures ranging in gradation from open to dense and tested with compactive efforts ranging from low to high, it can be concluded that some aggregates having a Los Angeles loss greater than the minimum commonly specified may, from the standpoint of degradation, be satisfactory materials especially if used in dense gradings subjected to low compactive effort.

5. Gradation of the mixture is the most important factor controlling degradation. As the gradation becomes denser, degradation decreases. The magnitude of this decrease is much greater than that brought about by changes in other variables. Soft or weak materials with high Los Angeles values can produce much less degradation than hard and strong materials if the former are used in dense-graded mixtures and the latter in open mixtures. Therefore, from a degradation point of view, dense-graded mixtures offer the best use of local aggregates with high Los Angeles values.

6. Increase in compactive effort results in increase in degradation of the mixture regardless of the form of this increase in effort, but degradation is more susceptible to change in magnitude of load than to change in repetition of load. The rate of change in degradation is high during the initial part of the application of compactive effort, and thereafter becomes less as the compactive effort is increased.

7. When the degradation of rounded particles is compared with that of angular particles of the same kind of aggregate, the rounded aggregate can be expected to produce less degradation because of a reduction of that portion of degradation which is due to wear. Use of rounded material will be helpful in reduction of degradation providing its use does not impair other properties of the mixtures.

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A Study of Impact Versus Conventional Mixing of Asphalt Concrete

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•DURING the summer of 1961, a West German manufacturer of impact mixing equipment approached the Ohio Highway Department to try impact mixing of asphalt concrete. In this approach, the manufacturer offered to bear all of the expense of the conversion, of an Ohio asphalt plant if the Highway Department would make a comparative study with the conventional method of mixing.

The manufacturer of the impact mixing equipment claimed several advantages would accrue by use of this method of mixing. The principal advantages claimed were:

1. Shorter mixing time.
2. More thorough and uniform coating of all of the particles of mineral aggregate.
3. A mix with better workability allowing it to be spread and compacted more satisfactorily.
4. A mix with greater stability for a given mix design.
5. A more durable mix.
6. Possible reduction of bitumen content.

After receiving this proposal the claimed advantages were studied and a review of the literature (1, 2, 3, 4, 5, 6, 7, 8) was made. Since there had been no significant change in mixing asphaltic concrete in Ohio from the time the material was first used, the decision was made to accept the proposal of the manufacturer for impact mixing equipment. Accordingly, plans were made to construct a project comparing impact and conventional methods of mixing.

In the impact mixing process (Fig. 1), the mixer shafts are rotated at a higher speed than in the conventional mixer. Also, the mixer tips are larger and set so that when the mixer is in operation the aggregates are tossed upward and fluffed so that they level off at about the top of the mixer for its full area. The asphalt cement is introduced into the mix under high pressure (240 psi) through three spray bars equipped with special nozzles. There are four nozzles on each spray bar directed downward and so located that the entire area of the mixer is blanketed evenly with a fog of bitumen. Coating of the aggregates is accomplished by the collision of the individual aggregate particles with the finely divided particles of bitumen. Thus, the process derives the name impact mixing. It is claimed the films of bitumen are continuous on each aggregate particle including all of the minus 200 mesh. There is no dry mixing of the aggregates. As soon as the aggregates are dumped into the mixer the bitumen starts to flow and the mixing is complete as soon as all of bitumen has been introduced. The mixing time varies with the amount of bitumen required by the mixes. In the case of mixes produced for this study, the mixing time varied from 25 sec for the lower asphalt contents to 31 sec for the higher asphalt contents.

PLANNING THE PROJECT

To make a valid comparison of the impact and conventional mixes in service, a pavement was selected for resurfacing that was in a uniform state of wear by traffic and the elements. This pavement was 9-in. thick, 24-ft wide reinforced portland cement concrete on 6 in. of classified subbase course. The subgrade soil in shallow cuts and fills was predominately BPR A-6 classification, a silty clay with some sand. The compaction requirements were a minimum of 100 percent for the top 6 in. of subgrade throughout the project and 95 percent for all embankment under the top 6 in. The compaction

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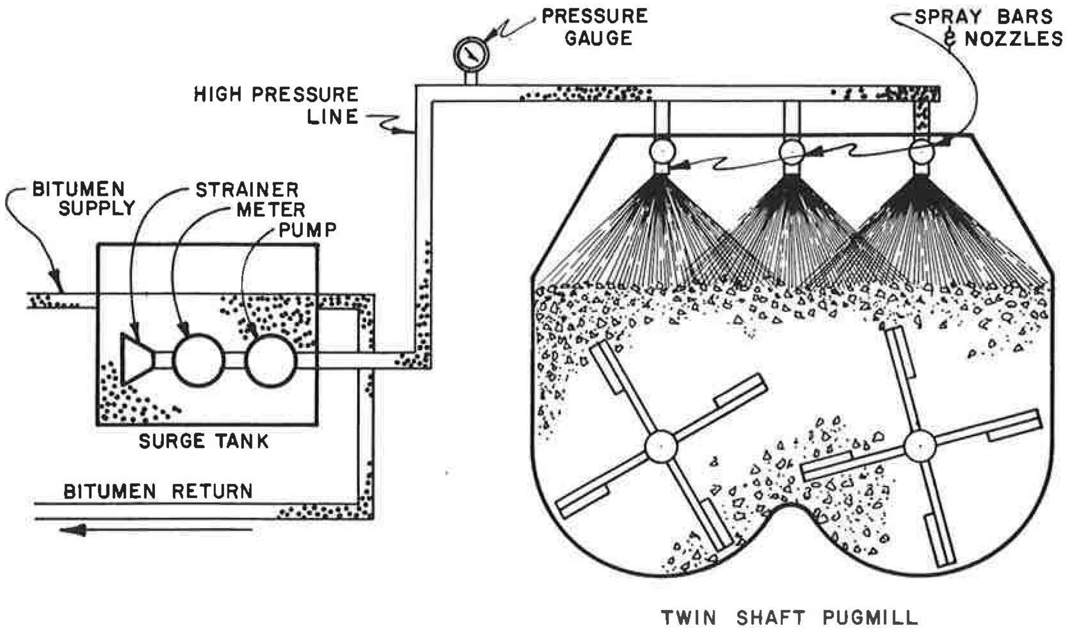


Figure 1. Bituminous concrete impact mixing system.

requirements were in accordance with AASHO Designation: T99-57 Method A.

The resurfacing for the study was of two course construction. The leveling course was set up for $1\frac{1}{4}$ in. of Ohio T-35 Type A asphalt concrete. The surface course was set up for $1\frac{1}{4}$ in. of Ohio T-35 Type C asphalt concrete. There were available near the location of the project both crushed limestone and 40 percent crushed gravel. In order to expand the experience with the impact mixing process it was decided to use both types of aggregates.

Because of the method of introduction of asphalt in the impact mixing process, there was fear of a more rapid rate of hardening of the asphalt than in the conventional method of mixing. To determine the rate of hardening it was decided to produce all of the mixes at three temperatures — 260, 280, and 300 F.

The possibility of being able to use a lesser amount of asphalt in the impact mixing process was another factor in planning the project. It was decided to produce all of the mixes at three asphalt contents: the normal contents for the mix based on past experience, 0.4 percent above, and 0.4 percent below the normal. Because a serviceable pavement was desired, it was decided that no large increase or decrease would be used in the asphalt content of the mixes.

With two aggregates, two mixes, three temperatures for each mix and three asphalt contents as variables, a total of 36 test sections were necessary if surface course mixes were superimposed on leveling mixes having identical variables. The pavement selected was the westbound lanes of US 40 extending 8.7 miles from Kirkersville to Reynoldsburg. It was desired to keep the asphalt plant in continuous operation once it was started in the morning. A total 8.7-mi project length allowed seventeen 0.5-mi test sections in each 12 ft lane and one 0.2-mi test section in each lane, a total of 36 sections in both lanes. This section length would permit the change from one temperature to another temperature or to a mix with another asphalt content in a two to three load transitional zone without interruption of plant production. Figure 2 shows the layout of the test sections of the project. The high asphalt content mixes were placed in the passing lane.

a Saybolt-Furol viscosity of approximately 100 sec at 300 F. Inasmuch as this viscosity was suitable for conventional mixing and the manufacturer of the impact mixing equipment felt it was nearly ideal for impact mixing, it was decided to maintain the asphalt cement at 300 F at the point of introduction into the mixes.

A unit price contract was negotiated with another contractor in the area to place and finish the mix. A conventional Barber-Greene paver was used in spreading. For the hourly tonnage produced, Ohio specifications require the use of three conventional steel-wheel rollers for compaction. The rollers used were a 10-ton three-wheel roller for breakdown, and two 8- to 12-ton variable-weight tandem rollers for intermedial and final rolling.

TEST PROCEDURE

The testing program was designed to insure close control of mix variables during production and to permit molding stability specimens and extraction and recovery of asphalt cement during the day in which the material was produced. In addition to the regular plant laboratory facilities, space was required for extraction and recovery apparatus and for gyratory compactor specimen molding equipment with a curing oven and a constant temperature bath.

Apparatus for the extraction, recovery, penetration and ductility tests was installed in a Highway Department trailer and moved to the plant site. This included a centrifuge for extraction of the asphalt and one for dust separation, 12 primary and 12 secondary distillators, a penetrometer and a ductility machine. The stability molding and testing equipment was housed in the regular laboratory building of the plant equipped for conventional mixing.

A system of sample numbering was devised which would furnish complete identification of the material and the point at which the sample was taken. The following is a key to the figures included in a typical sample number:

12A L III H

- H — Sampled from T = Truck at plant, H = Paver hopper, P = Pavement
- III — Number of the sample in the section
- L — L for leveling course, Type A and S for surface course, Type C
- 12A — Pavement section number (see Fig. 2).

The tests conducted on each sample load and the origin of the sample on which the test was made were as follows:

Tests	Sample Truck	Taken from Paver	Pavement
Aggregate gradation	X		
Asphalt content	X		
Penetration	X	X	X
Ductility	X	X	X
Marshall stability	X		
Unconfined compression	X		
Triaxial compression	X		
Pavement density			X

Subsequent samples were taken from a sample load as it was dumped into the paver hopper and again after the same material was compacted in the pavement. Three loads were sampled from each section except section 1 in which only one load was sampled. The sampled loads were equally spaced and were outside of the transitional zones between sections. Truck samples were taken immediately after loading, using a 15- 15- × 18- × 2-in. pan with the tests being performed on a percentage of the total quantity. The paver sample was taken from the hopper in a 8- × 10- × 2-in. pan and returned to the plant laboratory. Pavement samples were taken with a 4-in. diameter core drill immediately following the final compaction; density was determined in a mobile density control unit on the job. At the end of the day, the samples were transported to the plant laboratory for the indicated tests.

The standard procedures for bituminous concrete quality control, as outlined in the testing laboratory's "Manual for Plant Inspectors and Centrifuge Operators," were followed at both plants throughout the operation of the project. In addition to the daily grading of stockpiled aggregates and the hourly "carry-over" determinations, the plant inspection crew performed a complete mechanical analysis, using the field centrifugal extractor, on mix from each truck sample. A correction was applied to the bitumen determination based on the fines present in the liquid discharge from the centrifuge.

One Marshall specimen, 4-in. diameter by approximately 2½-in. height, was molded for each truck sample in the gyratory compactor at 120 psi and 60 gyrations at a 2^0 angle. The density of each specimen was obtained by reading the height of the specimen on a dial mounted on the gyratory compactor at the completion of compaction but before removal from the machine. This reading, along with the 4-in. mold diameter and the specimen weight, was used to compute the density. Stability and flow values for each specimen were determined using Marshall equipment in accordance with ASTM Designation: D-1559-60T.

One specimen, 4-in. diameter by approximately 4-in. height, was also made from each truck sample for testing in unconfined compression. Molding was the same as previously indicated. Specimen density was determined as for the Marshall specimens. The specimens were air cured 24 hr in a 140 F oven at the plant site; then encased in watertight rubber bags and immersed in a 77 F water bath for 4 hr; and then tested in unconfined axial compression to failure at a constant 0.2-in. per min rate of deformation. (Reference is made to AASHO Designation: T-167-60 with the given modifications.)

During the molding of the unconfined compression test specimens, additional data were taken concerning the mixture resistance to the gyratory movement. This was accomplished by measurement of the strain in the member through which the action is imparted to the mold. An SR-4 strain gage was attached to either side of the member. Instrumentation for exciting the gages and recording the circuit imbalance resulting from variations in strain produced a continuous graphic recording of the entire molding period.

A modified Abson procedure was employed for asphalt recovery. The asphalt was extracted by centrifuge from a 1,500-g portion of each sample using 1,250 ml of trichloroethylene as the solvent. Solvent was distilled from the extracted liquid at the rate of 20 ml per min until 200 ml of liquid remained. This portion was subjected to a centrifugal force of 770 gravities for 30 min to settle out most of the dust. The remaining solvent was distilled from the asphalt following the procedure in ASTM Designation: D-1856-61T. The recovered asphalt was tested for penetration and ductility.

The penetration test, AASHO Designation: T-49-53, was conducted at 25 C with a 100-g weight for 5 sec. The ductility test, AASHO Designation: T-51-44, was conducted at 25 C at a rate of 5-cm per sec and at 4 C at a rate of 1 cm per sec.

Triaxial tests were performed after the project was completed. A portion of each of the truck samples was set aside in closed containers for the triaxial specimens, which were made in the gyratory compactor following the same procedures as in the unconfined and Marshall specimens. Molds 3 in. in diameter were used, making 3-in. diameter by 6-in. high specimens. After molding, the samples were stored at room temperature. Before testing they were placed inside rubber bags and placed in a waterbath at 77 F for 4 hr. The samples were then placed in a Bureau of Public Roads triaxial cell and tested to failure under a 60-psi lateral air pressure and at a 0.3-in. per min axial rate of strain. There was no attempt to establish Mohr envelopes for the specimens.

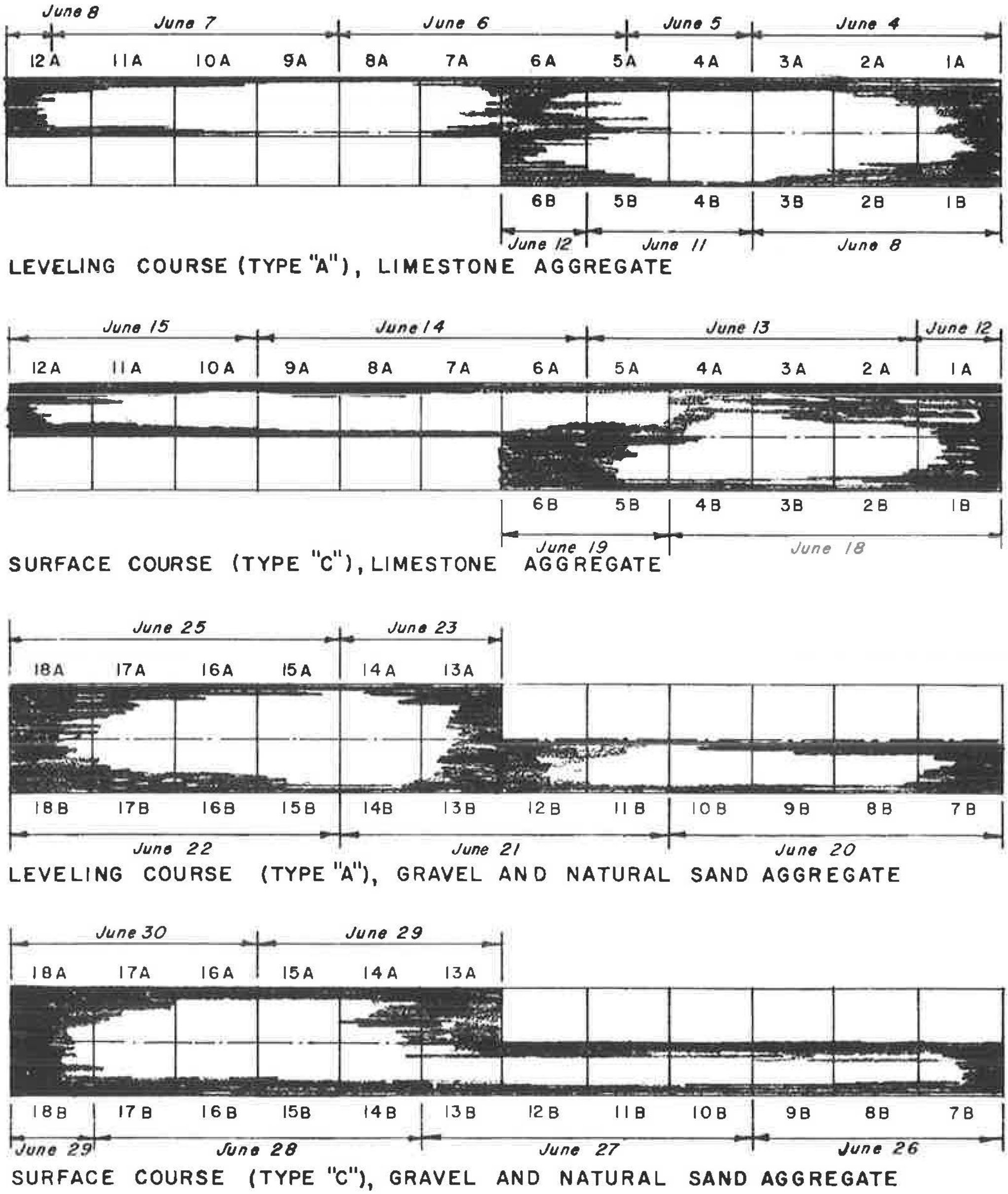


Figure 4.

PROGRESS AND WEATHER

Production of mix started on June 4, 1962, and was completed on June 30, 1962. Approximately 8,000 tons each were produced by both the impact and standard method of mixing.

Figure 4 shows the sequence of production and placement in the test sections on the road. All paving was done from East to West or from right to left in the diagrams. Table 2 gives climatological data for these working days. The observations at 8:00 AM, 12:00 Noon and 4:00 PM were taken from the U.S. Weather Bureau report for June 1962

TABLE 2

CLIMATOLOGICAL DATA FOR PROJECT WORKING DAYS

U.S. Weather Bureau Observations at Municipal Airport, Columbus, Ohio

DAY (JUNE 1962)	HOUR	SKY COVER TENTHS	DRY BULB (°F)	WET BULB (°F)	REL. HUM. (%)	WIND DIR.	WIND SPEED (KNOTS)	DAY (JUNE 1962)	HOUR	SKY COVER TENTHS	DRY BULB (°F)	WET BULB (°F)	REL. HUM. (%)	WIND DIR.	WIND SPEED (KNOTS)
4	08	10	69	67	90	SE	7	19	08	10	74	67	69	NW	6
	12	9	80	73	72	SSE	7		12	9	82	68	49	W	11
	16	7	84	72	57	S	9		16	5	86	69	40	NW	12
5	08	10	69	66	87	SSW	5	20	08	5	65	60	75	NNE	8
	12	8	77	70	69	W	7		12	5	74	63	54	NNW	11
	16	8	79	70	65	WNW	11		16	4	77	65	50	N	12
6	08	10	67	64	84	NNE	3	21	08	0	63	58	73	E	5
	12	10	71	65	73	ENE	9		12	0	74	61	48	ESE	6
	16	4	77	67	58	ENE	8		16	2	81	64	37	N	8
7	08	1	67	58	59	E	4	22	08	2	71	61	57	S	8
	12	3	80	66	47	ENE	12		12	3	83	65	36	S	9
	16	9	82	67	46	ESE	8		16	10	84	67	40	SSW	11
8	08	8	68	61	68	ESE	7	23	08	3	74	70	82	S	6
	12	7	81	70	56	ESE	8		12	10	84	72	55	SSW	5
	16	8	85	72	51	ESE	10		16	9	76	71	76	NNE	7
11	08	10	74	69	79	SSW	8	25	08	6	71	66	79	-	0
	12	10	74	70	84	-	0		12	8	80	66	45	NE	3
	16	10	76	71	79	S	14		16	10	82	67	44	N	7
12	08	10	67	64	87	S	3	26	08	6	72	65	68	-	0
	12	10	75	68	71	NNW	6		12	8	83	68	44	NE	4
	16	10	68	62	73	N	15		16	4	86	69	42	NNE	9
13	08	10	61	56	72	N	8	27	08	0	68	61	68	ENE	7
	12	10	63	59	78	N	10		12	0	83	70	51	ENE	6
	16	10	65	59	70	N	10		16	2	86	69	40	ENE	7
14	08	10	60	57	84	NNW	9	28	08	3	71	64	68	SSE	4
	12	10	66	61	76	WNW	6		12	3	86	68	39	WSW	4
	16	8	71	62	61	NNW	6		16	3	87	69	39	ESE	6
15	08	0	64	59	75	-	0	29	08	7	70	65	79	S	3
	12	3	75	63	52	ENE	6		12	4	84	66	38	SSW	4
	16	4	79	65	45	ENE	7		16	4	86	68	39	SE	6
18	08	0	74	66	66	S	6	30	08	1	73	65	64	-	0
	12	0	89	69	36	WSW	11		12	2	87	67	34	W	6
	16	3	94	72	33	SW	10		16	7	86	71	46	NNW	6

at the Columbus Municipal Airport. The project is approximately six miles southeast of the airport.

As a whole the weather was considered to be ideal for the paving work. Total precipitation recorded by the Weather Bureau during project working hours was 0. 29 in. which occurred on three days. On June 5, it rained during compaction of the last 600 ft of the leveling course on Section 4A and again during compaction of the first 500 ft of Section 5A. No test loads were affected in Section 4A. In Section 5A, the area included test load 5AL1. This area was situated at the bottom of a vertical curve where water collected during compaction. On June 11 rainfall occurred during the compaction of 700 ft of Section 4B leveling course which included test loads 4BL11 and 4BL111. On June 12, a light rainfall occurred during compaction of the first 300 ft of surface course on Section 1A. This did not include the test load 1AS1.

There were no major delays other than the above rainfall and the work progressed substantially as planned. The overall average rate of placing on the test sections was 81 tons per hr.

A traffic survey was made on the location in May and again in August 1962. Table 3 summarizes these surveys made at each end of the project.

RESULTS

On July 5, 1962, with the work completed, a meeting of those directly engaged in various phases of work was held to discuss impressions developed through observation and experience during production, placing, and testing of the various mixes. Partially tabulated results indicated the tested properties of the mixtures produced by the two

TABLE 3

24-HR TRAFFIC DENSITY WESTBOUND LANES

Date	Day of Week	Traffic Lane	Passenger Cars	Single-Unit Trucks	Tractor Semi-trailers	Full Trailer Combinations	Busses	Total Commercial	Total Vehicles
(a) West End of Project									
5-25-62	Fri.	Curb	3,873	372	547	23	36	978	4,851
5-25-62	Fri.	Median	883	29	13	4	3	49	932
8-10-62	Fri.	Curb	4,734	369	470	36	29	884	5,618
8-10-62	Fri.	Median	1,148	54	19	0	6	79	1,227
(b) East End of Project									
5-24-62	Thurs.	Curb	2,793	393	546	28	31	998	3,791
5-24-62	Thurs.	Median	713	46	65	1	7	119	832
8- 9-62	Thurs.	Curb	4,101	346	569	36	33	984	5,085
8- 9-62	Thurs.	Median	666	30	19	3	9	61	727

methods were quite similar. In general, this same impression was shared with regard to the handling, placing, and visual observation of the mixtures. Although the impact method was considered to produce material of satisfactory quality, analysis of all the data and observation of the pavement in service were considered necessary to determine possible superior qualities.

At this time, one advantage of the impact method, the mixing time requirement, was recognized and discussed. During production, it was found that the time required for introduction of the asphalt was the controlling factor in the mixing cycle time. This total time varied, of course, with the quantity of asphalt.

The mixing cycle time was also discussed in connection with needs for further investigation involving determination of the optimum relationship between the number of nozzles, temperature, pressure, and pugmill variables and directed toward further reduction in the cycle time. Mixing at elevated temperatures for paving during cold weather was also considered as an area for further study.

TEST RESULTS

Mechanical Analysis

The results of the mechanical analysis performed on material from each plant sample were combined by mix design and type of mixer. The results, single gradations, representative of each mix design are given in Tables 4 through 7.

Penetration of Recovered Asphalt

The penetration of the asphalt recovered from each sample is given in Tables 8 (limestone mixes) and 9 (gravel mixes). The results are arranged in two pairs of vertical columns. In each pair, adjacent samples are from test sections having identical mix designs. The samples are grouped by mixing temperature with the results averaged for each temperature group. The results of impact mixing may be compared with adjacent standard mixing results in each pair of columns.

This comparison is shown graphically in Figures 5 through 8 using the averages of the temperature groups expressed as percent of the original penetration. The

TABLE 4

AVERAGE PERCENT PASSING BY WEIGHT

Sieves	Imp. Mixing Sections			Std. Mixing Sections		
	7,8,9A	10,11,12A	4,5,6B	1,2,3A	4,5,6A	1,2,3B
1 in.	100.0	100.0	100.0	100.0	100.0	100.0
3/4 in.	97.5	97.8	97.9	96.9	97.2	97.5
1/2 in.	81.9	81.9	81.3	77.9	78.7	79.5
3/8 in.	67.8	68.1	68.4	63.1	66.5	67.2
1/4 in.	51.9	52.8	53.7	52.3	52.3	52.3
No. 4	45.9	45.6	45.9	45.6	46.2	45.6
No. 6	41.5	41.4	40.8	41.8	42.0	41.5
No. 8	35.4	35.9	35.1	36.2	36.5	36.2
No. 16	22.6	23.2	22.7	23.8	24.3	23.5
No. 30	14.6	15.1	15.1	15.9	16.1	15.1
No. 50	10.2	10.5	10.7	11.1	10.8	10.4
No. 100	7.3	7.6	7.6	7.7	7.2	7.3
No. 200	4.9	5.5	5.2	5.5	4.8	5.2
Bitumen	5.6	5.2	5.9	5.3	5.6	6.3

TABLE 5

AVERAGE PERCENT PASSING BY WEIGHT

Sieves	Imp. Mixing Sections			Std. Mixing Sections		
	13,14,15A	16,17,18A	7,8,9B	10,11,12B	13,14,15B	16,17,18B
1 in.	100.0	100.0	100.0	100.0	100.0	100.0
3/4 in.	99.2	98.8	98.5	98.8	98.8	98.9
1/2 in.	82.3	81.1	81.6	79.2	79.7	82.1
3/8 in.	68.6	70.7	69.3	68.7	72.2	70.2
1/4 in.	55.0	57.3	54.2	56.8	56.9	56.0
No. 4	46.8	47.9	43.7	47.3	48.1	47.4
No. 6	41.4	40.9	37.8	41.8	42.8	42.1
No. 8	38.1	36.9	33.4	37.7	38.1	37.9
No. 16	29.7	29.1	25.6	27.8	28.6	29.2
No. 30	19.3	19.9	15.6	16.8	17.6	18.6
No. 50	11.3	10.7	9.2	8.9	10.5	11.2
No. 100	6.5	5.3	5.7	5.0	6.1	6.2
No. 200	4.0	3.4	3.7	3.2	4.0	4.0
Bitumen	5.4	5.7	6.2	5.9	5.1	5.7

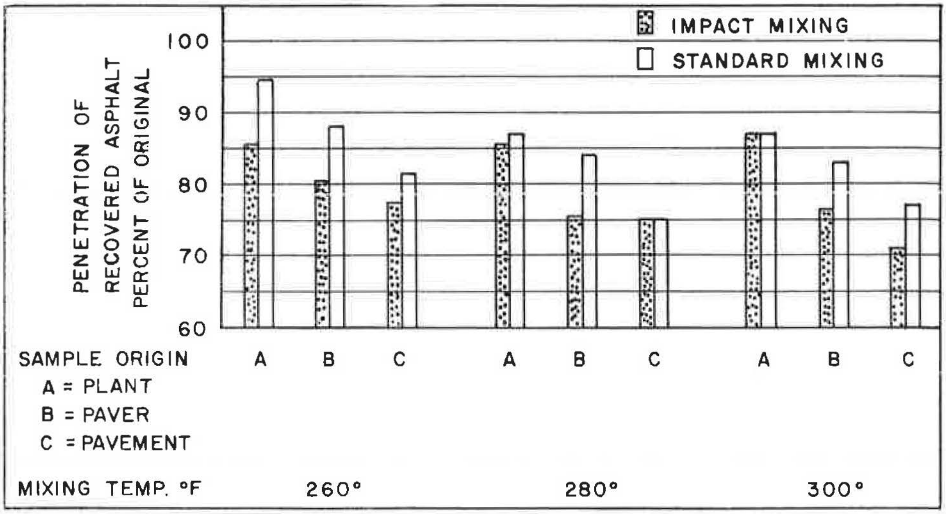


Figure 5. Penetration of asphalt recovered from Type A mixes, limestone aggregate.

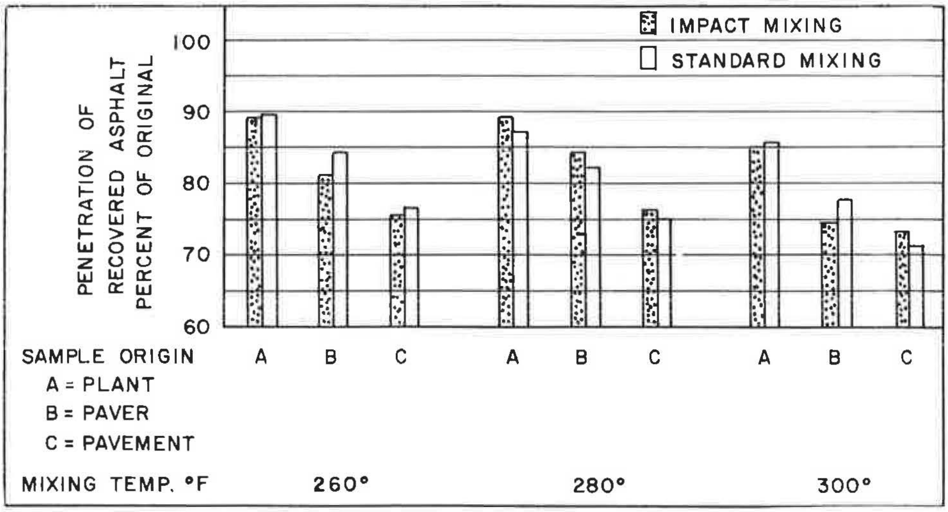


Figure 6. Penetration of asphalt recovered from Type C mixes, limestone aggregate.

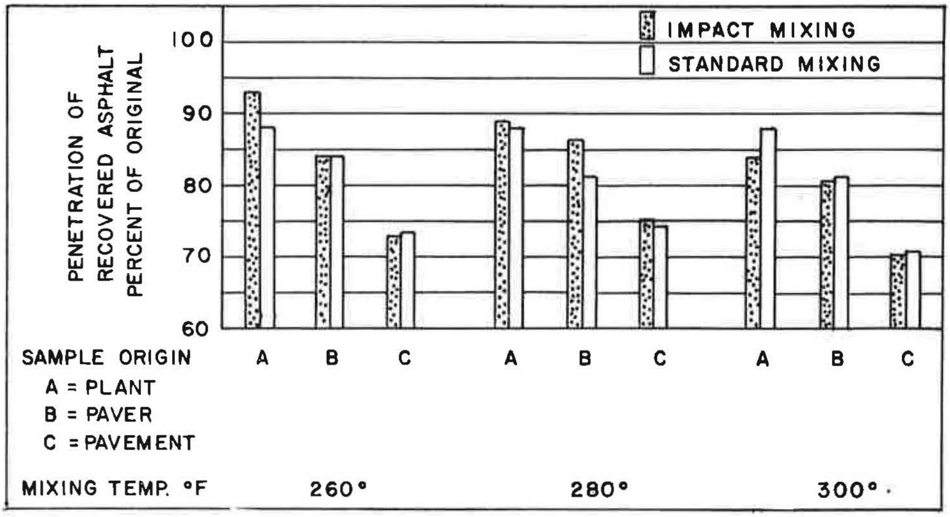


Figure 7. Penetration of asphalt recovered from Type A mixes, gravel aggregate.

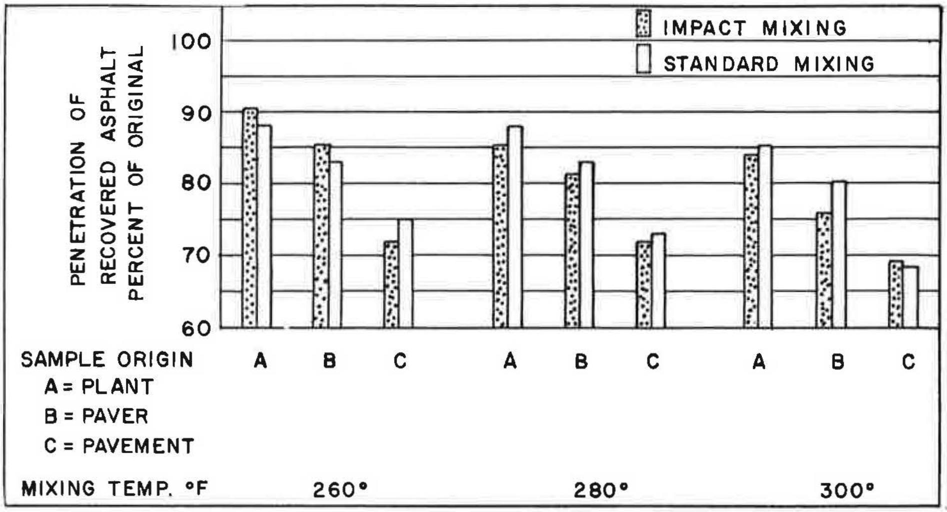


Figure 8. Penetration of asphalt recovered from Type C mixes, gravel aggregate.

original penetration was determined on a sample taken from the asphalt line at the beginning of production for each test section. This value averaged 76 for each type for standard mixing and, for impact mixing, 76 for Type A limestone, 74 for Type C limestone and Type A gravel and 75 for Type C gravel mixes.

There appears to be no significant difference in the two methods in this respect. This is particularly true of the comparison in Figures 6, 7, and 8. The data in Figure 5 may indicate some difference in penetration loss between the two methods; however, it is not consistent with the remaining data.

Ductility of Recovered Asphalt

At 25 C the ductility of all recovered asphalt samples was in excess of 150 cm. The ductility at 4 C for each sample is given in Tables 10 (limestone mixes) and 11 (gravel mixes). The results have been arranged as in Tables 8 and 9 for comparison. The ductility of the samples taken from the asphalt line was included since, unlike the penetration of these samples, the values are quite variable. This appears to be particularly true of the standard mixing samples in Table 10; however, occasional values appearing to be erratic occur throughout the data.

Laboratory and Pavement Density

Densities of laboratory specimens and pavement core samples are given in Tables 12, 13, 14, and 15. Average values of the tests in each section are given: an average of three laboratory specimen densities for each section and three pavement samples of three cores each. The tables are arranged with like sections adjacent horizontally to aid in making comparisons.

Analysis of the pavement density data reveals an overall greater density was attained in all the impact mixing sections except the Type A gravel aggregate mixes. Increased density at the same compactive effort is an indication that the impact mixes were more workable than the standard mixes. The percentage increase in density of the impact groups over the standard was found to average 1.8 percent for Type A limestone, 1.2 percent for Type C limestone, 0.6 percent for Type C gravel, and a decrease of 0.5 percent for Type A gravel.

The Type A gravel mixes were compacted in the pavement to a higher percentage of maximum density (and also of laboratory density) than the other mixes, leaving less room for the effect of a variation in workability to become apparent.

TABLE 10

RECOVERED ASPHALT DUCTILITY AT 4°C, LIMESTONE MIXES

Mixing Temp. (°F)	Impact Type A					Standard Type A					Impact Type C					Standard Type C				
	Sample	Origin				Sample	Origin				Sample	Origin				Sample	Origin			
		Asphalt Line	Plant	Paver	Pavement		Asphalt Line	Plant	Paver	Pavement		Asphalt Line	Plant	Paver	Pavement		Asphalt Line	Plant	Paver	Pavement
260	7A 1	11.8	11.8	11.2	16.2	1A 1	18.5	10.5	9.0	7.6	7A 1	11.0	8.8	11.5	7.8	1A 1	14.8	13.2	8.8	7.5
	2		14.8	11.2	14.2						2		9.5	8.8	10.5					
	3		13.2	11.2	10.5						3		8.8	8.8	10.0					
	12A 1		13.2	9.0	11.0	6A 1	27.0	10.8	10.5	10.2	12A 1	11.2	9.5	9.0	7.2	6A 1	14.0	10.2	8.2	8.2
	2		11.8	8.5	11.5	2		10.5	11.8	11.0	2		10.8	10.8	7.0	2		9.2	11.8	10.8
	3		12.0	9.2	9.0	3		10.8	13.8	11.5	3		11.0	9.5	7.2	3		9.5	7.5	16.8
	6B 1	13.2	10.0	9.8	12.5	1B 1	30.8	9.8	17.5	16.0	6B 1	9.0	9.2	7.2	6.0	1B 1	39.8	32.8	31.5	8.0
	2		13.0	8.8	7.8						2		6.5	8.5	5.8					
	3		12.0	7.8	7.5						3		6.5	7.2	6.2					
280	8A 1	14.8	9.8	20.5	8.5	2A 1	26.5	13.5	10.0	7.0	8A 1	11.2	11.2	8.0	9.5	2A 1	15.0	10.8	9.8	9.5
	2		10.8	9.8	8.2	2		10.8	8.5	5.6	2		11.8	8.2	9.2	2		9.5	7.5	8.0
	3		14.2	8.8	7.5	3		9.9	9.3	6.0	3		11.0	8.8	8.8	3		9.5	10.2	9.2
	11A 1	14.8	9.2	10.2	7.0	5A 1	16.8	26.2	17.0	9.5	11A 1	12.0	9.8	8.5	7.0	5A 1	49.0	9.8	19.5	8.8
	2		12.5	7.5	11.8	2			18.2	11.0	2		9.0	8.5	7.2	2		19.5	23.5	8.2
	3		14.0	8.2	9.0	3		17.8		13.8	3		10.5	7.2	7.2	3		10.2	15.2	9.8
	5B 1		9.5	20.0	8.2	2B 1	39.5	11.5	8.8	8.8	5B 1	11.0	8.5	7.5		2B 1	11.0	9.2	9.0	8.0
	2		9.0	10.0	9.5	2		9.8	9.8	9.5	2		6.5	8.8	6.2	2		10.5	9.5	6.8
	3		12.5	9.2	9.2	3		12.5	9.2	9.5	3		6.5	11.5	6.5	3		10.5	9.8	6.2
300	9A 1	20.2	9.8	20.8	7.5	3A 1		11.5	8.5	7.5	9A 1	13.5	24.5	7.0	8.0	3A 1	17.8	11.5	10.2	7.8
	2		12.5	8.2	8.0	2		9.8	10.5	7.5	2		23.8	7.2	7.0	2		10.2	8.0	9.0
	3		12.5	9.2	7.0	3		10.8	14.2	8.8	3		10.8	15.0		3		10.8	11.2	10.2
	10A 1	14.5	11.0	7.8	8.0	4A 1	14.0	9.8	11.0	11.8	10A 1	25.8	12.0	7.0	7.0	4A 1	13.2	9.2	26.5	17.0
	2		9.2	9.5	6.5	2		10.2	10.8	8.0	2		9.0	8.0	6.8	2		8.2	7.2	12.0
	3		12.5	6.8	7.2	3		10.8	9.0	8.2	3		10.0	9.8	6.5	3		11.2	9.8	9.2
	4B 1	12.2	11.0	8.2	9.5	3B 1	17.5	9.2	12.5	13.8	4B 1	12.5	9.0	10.0	7.5	3B 1	10.0	9.8	7.0	5.5
	2		11.0	8.5	12.5	2		9.5	10.8	10.0	2		9.8	6.2	7.2	2		9.5	8.8	6.5
	3		12.5	8.8	7.5	3		10.8	8.2	13.8	3		9.2	6.8	6.2	3		9.8	8.5	6.5

TABLE 11
RECOVERED ASPHALT DUCTILITY AT 4°C, GRAVEL MIXES

Mixing Temp. (°F)	Impact Type A Sample	Impact Type A Origin: Asphalt Line	Plant	Paver	Pavement	Standard Type A Sample	Standard Type A Origin: Asphalt Line	Plant	Paver	Pavement	Impact Type C Sample	Asphalt Line	Plant	Paver	Pavement	Standard Type C Sample	Asphalt Line	Plant	Paver	Pavement
260	13A 1	17.0	15.8	13.5	5.0	12B 1	11.0	10.8	8.8	7.0	13A 1	9.2	8.8	7.5	11.0	12B 1	9.5	7.8	8.5	7.8
	2		10.5	10.5	4.5	2		9.8	9.2	6.8	2		8.8	7.5	9.8	2		8.0	6.2	8.0
	3		12.2	9.0	4.8	3		10.5	8.2	7.2	3		9.5	7.8	10.2	3		8.8	6.0	7.8
	18A 1	10.5		7.2	7.8	13B 1	23.8	10.2	13.0	6.5	18A 1	9.2	7.8	7.2		13B 1	13.0	6.8	5.8	8.5
	2		8.5	7.2	8.2	2		8.8	8.8	6.5	2		8.5	6.8	8.8	2		7.2	5.0	7.5
	3		9.5	7.2	7.5	3		9.2	9.8	6.2	3		8.2	6.8	8.2	3		7.0	5.8	8.2
	7B 1	8.2	10.0	9.8		18B 1	9.2	8.2	7.0	5.5	7B 1	7.5	7.0	5.8	7.5	18B 1	7.2	7.5	8.2	10.8
	2		10.2	11.2	9.5	2		7.5	7.0	6.2	2		6.8	6.2	7.0	2		8.5	10.2	10.5
	3		9.5	10.2		3		8.0	7.2	5.8	3			7.5	7.2	3		8.5	8.5	8.5
280	14A 1	16.5	12.8	11.5	4.8	11B 1	10.0	7.8	7.5	5.8	14B 1	11.8	8.0	7.0	7.8	11B 1	10.5	7.8	6.5	6.8
	2		9.5	9.0	5.0	2		8.8	9.0	6.5	2		8.8	7.0	10.2	2		7.8	6.8	6.8
	3		12.5	9.2	5.0	3		8.2	7.8	4.8	3		8.0	7.8	8.8	3		8.0	6.0	7.0
	17A 1	9.0	6.5	5.8	5.5	14B 1	15.5	9.5	8.5	6.2	17A 1	10.8	8.8	7.8	8.8	14B 1	8.2	7.8	6.5	8.2
	2		7.0	7.5	5.5	2		13.2	10.0	6.0	2		8.2	7.5	9.0	2		8.2	6.8	6.8
	3		7.2	6.5	5.2	3		13.8	13.5	6.2	3		8.0	6.8	8.8	3		8.0	7.2	8.0
	8B 1	18.0	8.2	10.5	9.2	17B 1	8.2	8.5	5.5	6.2	8B 1	8.8	7.8	6.8	8.5	17B 1	8.8	7.8	6.8	9.0
	2		8.0	11.5	8.5	2		7.8	5.5	5.8	2		7.5	7.8	8.2	2		6.0	7.5	9.8
	3		8.5	10.0	8.8	3		7.0	6.0	6.0	3		8.5	6.0	8.5	3		6.8	7.0	10.0
300	15A 1	18.5	7.0	6.8	5.2	10B 1	8.2	8.0	6.2	5.5	15A 1	9.5	9.0	6.5	9.5	10B 1	11.2	7.0	6.2	7.8
	2		7.2	6.2	5.0	2		7.2	6.2	8.0	2		9.5	6.8	9.8	2		7.2	6.5	7.8
	3		8.5	7.0	5.2	3		9.2			3		8.5	7.0	8.8	3		6.8	6.5	7.8
	16A 1	8.2	7.0	6.8	7.0	15B 1	10.5	6.2	7.0	6.0	16A 1	10.5	6.5	8.8	7.2	15B 1	9.5	7.2	7.0	9.2
	2		5.8	7.8	7.0	2		6.8	6.8	5.5	2		6.8	8.2	12.0	2		8.2	6.8	9.5
	3		9.2	6.2	6.8	3		6.8	5.5	6.2	3		6.5	8.2	12.2	3		8.0	7.2	9.0
	9B 1	9.0	6.5	5.8	5.5	16B 1	10.5	9.2	5.2	5.8	9B 1	12.2	6.2	6.5	7.8	16B 1	9.2	7.8	6.5	8.5
	2		7.0	7.5	5.5	2		8.5	7.5	5.0	2		8.0	6.2	7.5	2		6.5	5.8	10.2
	3		7.2	6.5	5.2	3		7.0	9.5	5.5	3		6.0	6.0	7.8	3		6.5	5.2	

Laboratory Stability

Tables 16 through 19 give results of stability tests. Average values of the tests on three specimens in each section are given.

The indication of greater workability in the impact mixes based on increased density is further substantiated by the laboratory stability data. The results show the impact mixes had lower stabilities in both Marshall and unconfined compression tests. The Marshall flow values, however, appear to be nearly equal.

Molding Resistance

The results of molding resistance are shown in Figures 9 through 19. The horizontal axis represents the number of 2^0 gyrations on a log scale. The vertical axis represents strain in microinches per inch on an arithmetic scale. Data have been plotted for the Type A and Type C mixes at the 280 F mixing temperature for three asphalt contents for each of the aggregate types. The molding cycle is stopped at 60 gyrations as it had been established in previous work that the density of the mixes at this point correlated with the maximum density produced by traffic.

In the case of the mixes with limestone and limestone sand the surface characteristics of the aggregates at the asphalt contents used determine to a large extent the stability characteristics of the mixes. Figures 9 and 10 are for Type A limestone mixes at asphalt contents of 5.9 and 5.5 percent. At both asphalt contents the impact mixes have greater resistance to molding than the standard mixes. However, this is rather slight and probably has little significance. Figures 11 through 13 represent Type C mixes for limestone at asphalt contents of 6.8, 6.4 and 6.0 percent. At an asphalt content of 6.8 percent the impact mix has less resistance to molding than the standard mix, at 6.4 percent asphalt the molding resistance is almost the same, and at 6 percent asphalt the molding resistance of the impact mix is significantly greater.

These data seem to be at variance with the stability and density data. The impact mixes were the more workable in every case, resulting in higher density in the pavement. These data may be reflecting the condition of or the characteristics of the asphalt film in the impact mixes.

In the case of the mixes with 40 percent crushed gravel and natural sand, the surface characteristics of the aggregates at the asphalt contents used are not of as much influence in determining stability characteristics of the mixes as with the limestone

TABLE 12

SUMMARY OF PAVEMENT CORE SAMPLE AND LABORATORY SPECIMEN DENSITIES
TYPE A MIXES, LIMESTONE AND LIMESTONE SAND AGGREGATE

Impact Mixer

Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.
7A	5.5	5.8	260	260	143.2	154.3	92.8	147.9 149.6	96.8 95.7
8A	5.5	5.4	280	280	139.7	154.3	90.5	146.5 149.8	95.4 93.2
9A	5.5	5.5	300	300	144.2	154.3	93.4	148.9 149.9	96.8 96.1
4B	5.9	5.9	300	300	143.5	153.3	93.6	148.6 150.1	96.6 95.6
5B	5.9	6.0	280	280	143.4	153.3	93.5	147.9 149.4	96.9 96.0
6B	5.9	5.7	260	260	142.8	153.3	93.2	148.3 149.2	96.3 95.7
10A	5.2	5.3	300	300	143.1	156.0	91.7	146.1 148.4	97.9 96.4
11A	5.2	5.3	280	280	141.5	156.0	90.7	146.6 148.9	96.5 95.0
12A	5.2	5.2	260	260	140.0	156.0	89.8	148.0 149.1	94.6 93.9

Standard Mixer

Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.
1A	5.5	5.2	260	260	142.2	154.0	92.3	147.6 149.7	96.3 94.3
2A	5.5	5.2	280	280	139.7	154.0	90.8	147.3 149.4	94.9 93.5
3A	5.5	5.4	300	300	137.7	154.0	89.4	147.1 149.1	93.6 92.3
4A	5.9	5.6	300	300	141.5	153.9	91.9	147.9 149.3	95.7 94.8
5A	5.9	5.3	280	280	141.1	153.9	91.7	147.1 148.8	95.9 94.8
6A	5.9	5.9	260	260	141.1	153.9	91.7	148.6 149.8	94.9 94.2
1B	6.3	6.8	260	260	142.2	152.0	93.6	148.8 149.0	95.5 95.4
2B	6.3	6.0	280	280	143.5	152.0	94.4	147.8 148.4	97.1 96.7
3B	6.3	6.2	300	300	143.0	152.0	94.1	147.7 148.9	96.7 96.0

[1]Upper value is density (in pcf) of Marshall specimen; lower, density (in pcf) of unconfined compression specimen.

TABLE 13

SUMMARY OF PAVEMENT CORE SAMPLE AND LABORATORY SPECIMEN DENSITIES
TYPE A MIXES, CRUSHED GRAVEL AND NATURAL SAND AGGREGATE

Impact Mixer

Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.
7B	6.1	6.1	260	260	144.7	153.3	94.4	148.6 149.7	97.4 96.7
8B	6.1	6.2	280	280	144.9	153.3	94.5	149.4 152.1	97.0 95.2
9B	5.7	5.6	300	305	143.6	154.1	93.2	149.4 150.0	96.1 95.7
16A	5.7	5.5	300	300	143.8	152.7	94.2	148.4 149.6	97.0 96.2
17A	5.7	5.7	280	280	145.1	152.7	95.0	148.6 149.4	97.6 97.0
18A	5.7	5.6	260	260	144.3	152.7	94.5	148.5 149.5	97.2 96.5
13A	5.3	5.5	260	260	140.3	154.3	90.9	149.6 150.0	93.8 93.6
14A	5.3	5.2	280	280	142.5	154.3	92.4	147.7 149.7	96.5 95.2
15A	5.3	5.4	300	300	144.3	154.3	93.5	148.0 149.3	97.5 96.6

Standard Mixer

Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.
12B	6.1	5.9	260	260	146.1	151.6	96.4	149.1 149.6	98.0 97.6
11B	6.1	6.0	280	280	139.0	151.6	94.1	148.7 149.0	96.0 95.8
10B	6.1	5.9	300	300	144.3	151.6	95.1	148.2 148.5	97.4 97.2
16B	5.7	5.7	300	300	144.6	153.4	94.2	148.3 149.7	97.6 96.6
17B	5.7	5.7	280	280	145.0	153.4	94.5	149.1 150.1	97.2 96.6
18B	5.7	5.8	260	260	145.9	153.4	95.1	148.9 148.7	98.0 97.5
13B	5.3	5.2	260	260	143.8	154.0	93.4	148.4 149.7	96.9 96.1
14B	5.3	4.8	280	280	142.3	154.0	92.4	148.1 149.1	96.1 95.5
15B	5.3	5.2	300	295	145.5	154.0	94.5	148.4 150.0	98.0 97.0

[1]Upper value is density (in pcf) of Marshall specimen; lower, density (in pcf) of unconfined compression specimen.

TABLE 14

SUMMARY OF PAVEMENT CORE SAMPLE AND LABORATORY SPECIMEN DENSITIES
TYPE C MIXES, LIMESTONE AND LIMESTONE SAND AGGREGATE

Impact Mixer										Standard Mixer									
Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.	Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.
7A	6.0	5.9	260	260	133.5	155.4	85.9	145.3 147.0	91.9 90.8	1A	6.0	5.8	260	260	138.0	155.3	88.9	146.3 147.1	94.3 93.8
8A	6.0	6.0	280	280	136.4	155.4	87.8	146.2 148.2	93.3 92.1	2A	6.0	5.9	280	280	136.2	155.3	87.7	147.4 149.7	92.4 90.9
9A	6.0	6.0	300	300	137.6	155.4	88.6	145.9 148.0	94.3 93.0	3A	6.0	5.5	300	300	134.0	155.3	86.3	145.4 148.0	92.2 90.6
10A	6.4	6.4	300	300	137.3	153.2	89.6	147.0 149.1	93.4 92.1	4A	6.4	6.2	300	300	135.0	154.0	87.7	147.0 148.8	91.8 90.7
11A	6.4	6.3	280	280	139.0	153.2	90.8	146.3 147.8	95.0 94.0	5A	6.4	6.4	280	280	134.7	154.0	87.5	146.5 147.8	92.0 91.2
12A	6.4	6.4	260	260	137.2	153.2	89.6	145.9 148.0	94.0 92.7	6A	6.4	6.3	260	260	133.9	154.0	87.0	146.4 147.7	91.7 90.6
6B	6.8	6.8	260	260	140.3	151.9	92.4	147.0 148.2	95.5 94.7	1B	6.8	6.4	260	260	135.5	152.0	89.1	145.5 147.3	93.1 92.0
5B	6.8	6.6	280	280	138.8	151.9	91.4	146.8 148.6	94.6 93.5	2B	6.8	6.8	280	280	139.5	152.0	91.8	147.2 147.7	94.8 94.5
4B	6.8	6.7	300	300	141.0	151.9	92.8	147.8 148.9	95.4 94.6	3B	6.8	6.5	300	300	138.4	152.0	91.0	146.6 148.2	94.4 93.3

[1]Upper value is density (in pcf) of Marshall specimen; lower, density (in pcf) of unconfined compression specimen.

TABLE 15

SUMMARY OF PAVEMENT CORE SAMPLE AND LABORATORY SPECIMEN DENSITIES
TYPE C MIXES, CRUSHED GRAVEL AND NATURAL SAND AGGREGATE

Impact Mixer										Standard Mixer									
Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.	Section	Design % AC	Actual % AC	Design Temp.(°F)	Actual Temp.(°F)	Pavement Core Density	Max. Density by Volumeter	Pavement % of Max.	Gyratory Compactor Density[1]	Pavement % of Gyr. Comp.
7B	6.6	6.6	260	260	140.8	150.0	93.9	147.5 147.4	95.5 95.6	12B	6.6	6.5	260	265	142.2	151.1	94.1	147.6 148.2	96.4 95.9
8B	6.6	6.4	280	280	142.5	150.0	95.0	148.1 148.2	96.2 96.1	11B	6.6	6.8	280	280	139.9	151.1	92.6	147.0 147.8	95.2 94.7
9B	6.6	6.4	300	300	143.0	150.0	95.3	147.9 148.4	96.7 96.3	10B	6.6	6.5	300	300	142.1	151.1	94.0	147.9 148.5	96.1 95.7
13A	5.8	5.9	260	255	138.6	152.4	90.9	146.5 147.3	94.6 94.1	13B	5.8	5.8	260	260	138.0	152.0	90.8	148.0 148.2	93.3 93.1
14A	5.8	5.8	280	280	139.0	152.4	91.2	146.2 147.0	95.0 94.6	14B	5.8	5.8	280	280	137.5	152.0	90.4	147.1 148.1	93.5 92.8
15A	5.8	5.8	300	295	140.4	152.4	92.1	144.7 147.6	97.0 95.1	15B	5.8	5.8	300	300	139.3	152.0	91.6	146.8 147.2	94.9 94.6
16A	6.2	6.2	300	300	140.3	152.9	91.8	144.8 147.7	96.9 95.0	16B	6.2	6.0	300	300	139.4	153.0	91.1	147.6 148.3	94.4 94.0
17A	6.2	6.1	280	280	140.6	152.9	92.0	145.2 147.6	96.9 95.3	17B	6.2	6.2	280	285	140.3	153.0	91.7	147.8 148.0	94.9 94.8
18A	6.2	6.1	260	260	141.1	152.9	92.3	145.4 148.0	97.0 95.3	18B	6.2	6.1	260	255	140.7	153.0	92.0	147.6 148.0	95.3 95.1

[1]Upper value is density (in pcf) of Marshall specimen; lower, density (in pcf) of unconfined compression specimen.

TABLE 16

SUMMARY OF LABORATORY STABILITY
TYPE C MIXES, LIMESTONE AND LIMESTONE SAND AGGREGATE

Impact Mixer									Standard Mixer							
Section	Marshall			Unconfined Compression		Triaxial Compression		% Asphalt Cement	Section	Marshall			Unconfined Compression		Triaxial Compression	
	Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)	Dens. (pcf)	Stabl. (psi)			Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)	Dens. (pcf)	Stabl. (psi)
7A	145.3	2,801	14	147.0	414	135.6	442	6.0	1A	146.3	3,012	13	147.1	399	136.4	465
8A	146.2	3,144	15	148.2	445	137.3	462	6.0	2A	147.4	3,288	18	149.7	405	138.6	480
9A	145.9	3,164	16	148.0	434	137.2	467	6.0	3A	145.4	3,134	17	148.0	458	137.4	469
10A	147.0	3,309	20	149.1	419	139.1	454	6.4	4A	147.0	3,341	17	148.8	425	138.1	491
11A	146.3	2,971	15	147.8	404	137.1	421	6.4	5A	146.5	3,263	19	147.8	372	137.0	447
12A	145.9	2,911	16	148.0	380	136.1	387	6.4	6A	146.4	2,700	17	147.7	375	136.5	463
6B	147.0	2,797	17	148.2	320	138.2	402	6.8	1B	145.5	2,700	16	147.3	359	135.6	373
5B	146.8	2,915	17	148.6	357	138.3	439	6.8	2B	147.2	3,104	17	147.7	395	136.6	393
4B	147.8	3,110	19	148.9	352	140.9	521	6.8	3B	146.6	3,105	17	148.2	406	137.4	439

TABLE 17

SUMMARY OF LABORATORY STABILITY
TYPE C MIXES, CRUSHED GRAVEL AND NATURAL SAND AGGREGATE

Impact Mixer									Standard Mixer							
Section	Marshall			Unconfined Compression		Triaxial Compression		% Asphalt Cement	Section	Marshall			Unconfined Compression		Triaxial Compression	
	Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)	Dens. (pcf)	Stabl. (psi)			Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)	Dens. (pcf)	Stabl. (psi)
7B	147.5	2,404	16	147.4	288	142.5	502	6.6	12B	147.6	2,451	15	148.2	332	142.2	510
8B	148.1	2,626	15	148.2	306	142.6	490	6.6	11B	147.6	2,763	15	147.8	374	141.1	511
9B	147.9	2,711	16	148.4	334	142.5	482	6.6	10B	147.9	2,694	14	148.5	339	142.1	501
13A	146.5	2,447	15	147.3	401	139.9	495	5.8	13B	148.0	2,588	15	148.2	349	142.0	517
14A	146.2	2,636	15	147.0	417	140.2	511	5.8	14B	147.1	2,811	14	148.1	403	141.2	499
15A	144.7	2,689	12	147.6	434	140.6	513	5.8	15B	146.8	3,007	14	147.2	421	141.0	530
16A	144.8	2,825	13	147.7	428	141.0	554	6.2	16B	147.6	2,859	13	148.3	426	141.9	595
17A	145.2	2,648	14	147.6	376	143.1	497	6.2	17B	147.8	2,510	16	148.0	308	144.0	568
18A	145.4	2,447	14	148.0	326	142.6	532	6.2	18B	147.6	2,643	14	148.0	339	142.6	569

TABLE 18

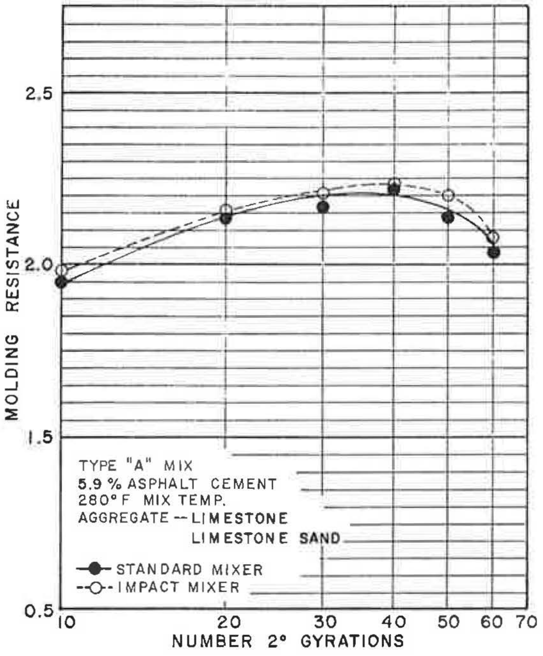
SUMMARY OF LABORATORY STABILITY
TYPE A MIXES, LIMESTONE AND LIMESTONE SAND AGGREGATE

Impact Mixer							Standard Mixer					
Section	Marshall			Unconfined Compression		% Asphalt Cement	Section	Marshall			Unconfined Compression	
	Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)			Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)
7A	147.9	2,983	18	149.6	358	5.5	1A	147.6	2,868	20	149.7	401
8A	146.5	2,905	18	149.8	400	5.5	2A	147.3	3,246	19	149.4	420
9A	148.9	3,302	21	149.9	397	5.5	3A	147.1	3,155	16	149.1	443
4B	148.6	3,261	21	150.1	367	5.9	4A	147.9	2,958	17	149.3	414
5B	147.9	2,757	20	149.4	340	5.9	5A	147.1	2,936	17	148.8	411
6B	148.3	2,756	20	149.2	315	5.9	6A	148.6	2,691	17	149.8	344
10A	146.1	3,145	19	148.4	429	5.2/6.3	1B	148.8	2,450	20	149.0	281
11A	146.6	3,013	18	148.9	440	5.2/6.3	2B	147.8	2,754	19	148.4	318
12A	148.0	3,037	16	149.1	439	5.2/6.3	3B	147.7	3,383	20	148.9	334

TABLE 19

SUMMARY OF LABORATORY STABILITY
TYPE A MIXES, CRUSHED GRAVEL AND NATURAL SAND AGGREGATE

Impact Mixer							Standard Mixer					
Section	Marshall			Unconfined Compression		% Asphalt Cement	Section	Marshall			Unconfined Compression	
	Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)			Dens. (pcf)	Stabl. (lb)	Flow (0.01 in.)	Dens. (pcf)	Stabl. (psi)
7B	148.6	2,385	19	149.7	251	6.1	12B	149.1	2,495	16	149.6	295
8B	149.4	2,145	19	152.1	249	6.1	11B	148.7	2,539	16	149.0	302
9B	149.4	2,539	17	150.0	372	5.7/6.1	10B	148.2	2,547	15	148.5	275
16A	148.4	2,742	13	149.6	394	5.7	16B	148.3	2,718	13	149.7	351
17A	148.6	2,601	14	149.4	348	5.7	17B	149.1	2,426	17	150.1	329
18A	148.5	2,605	15	149.5	336	5.7	18B	148.9	2,638	15	148.7	345
13A	149.6	2,244	16	150.0	342	5.3	13B	148.4	2,594	15	149.7	404
14A	147.7	2,699	15	149.7	389	5.3	14B	148.1	2,830	15	149.1	438
15A	148.0	3,040	13	149.3	423	5.3	15B	148.4	2,869	16	150.0	379



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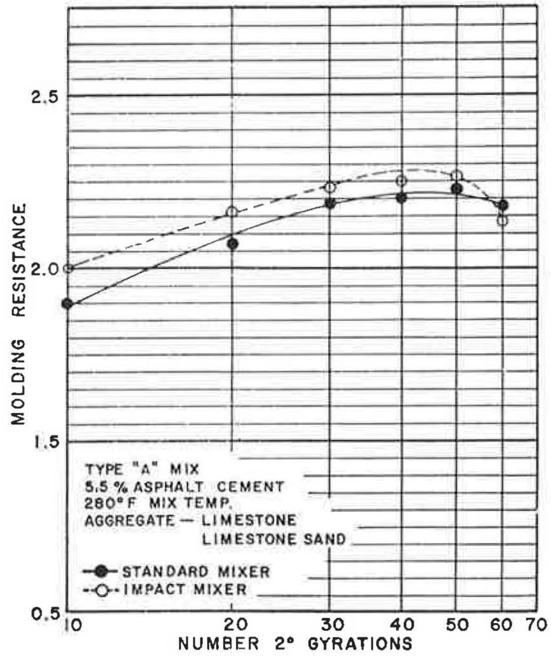

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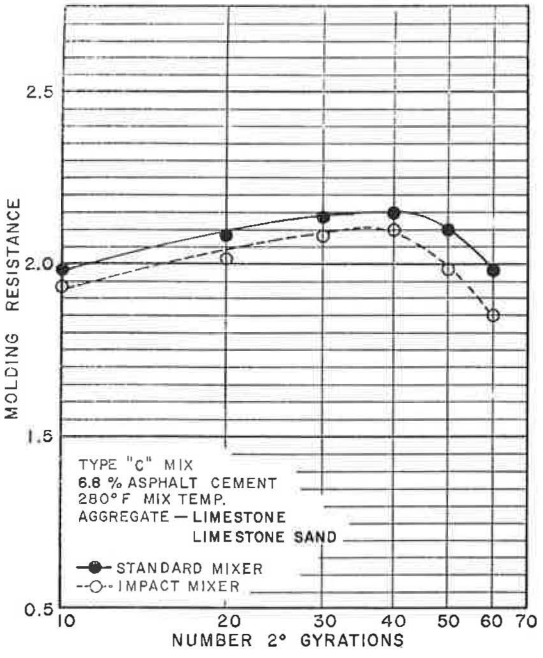

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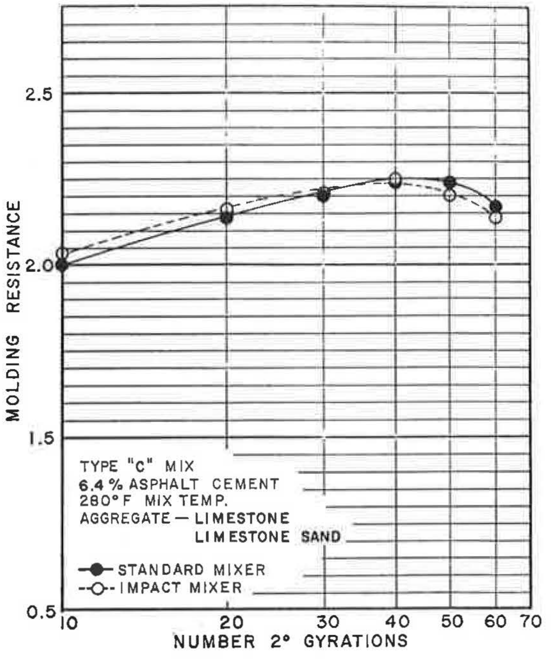

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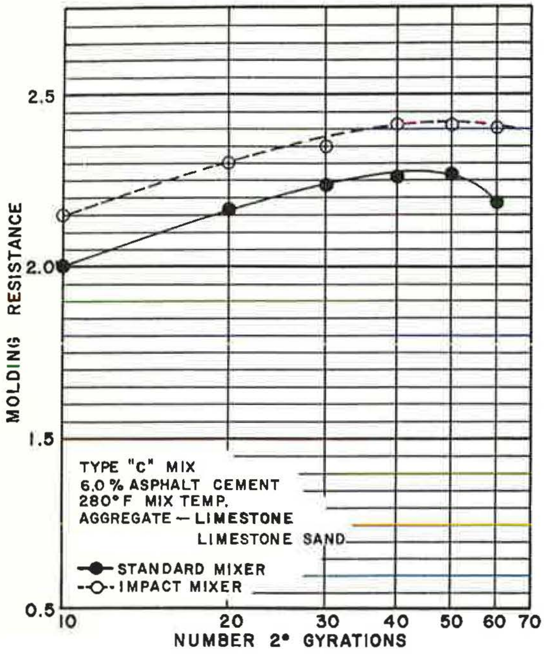

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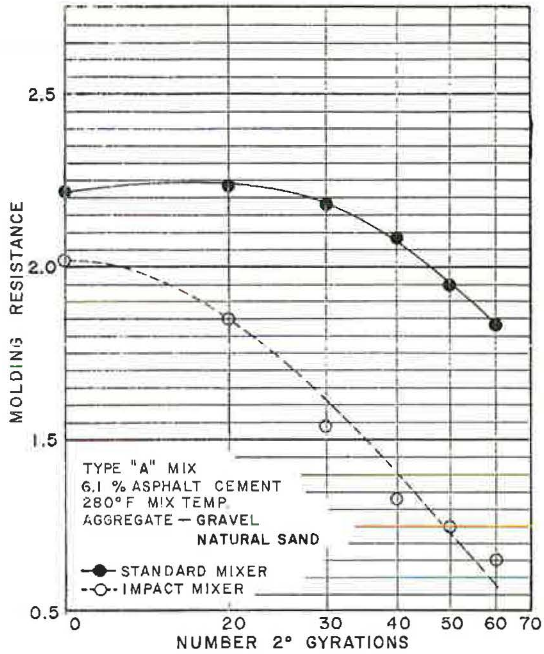

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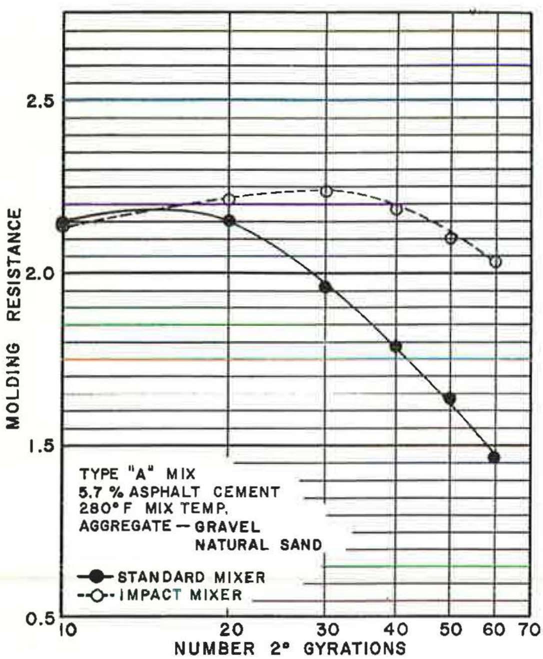

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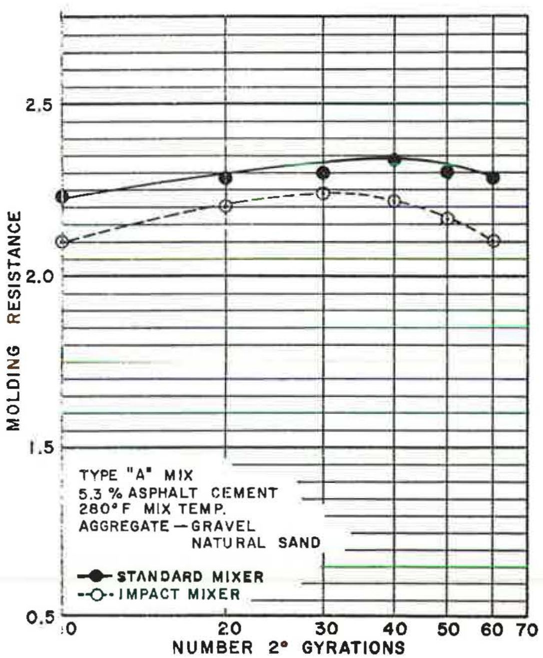

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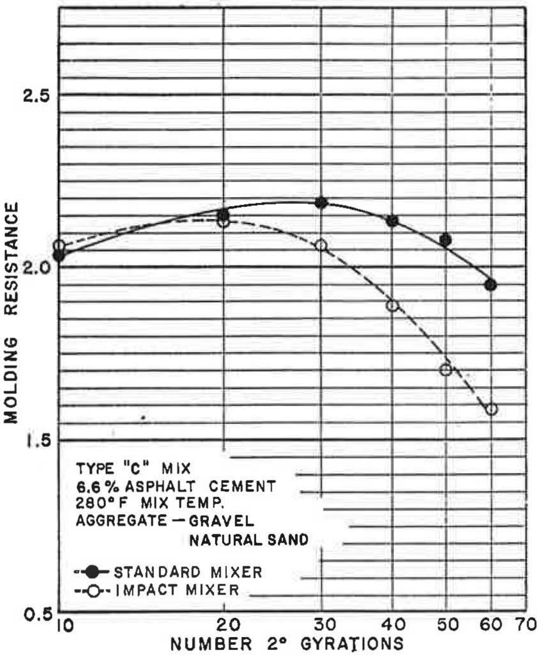

Figure 17.

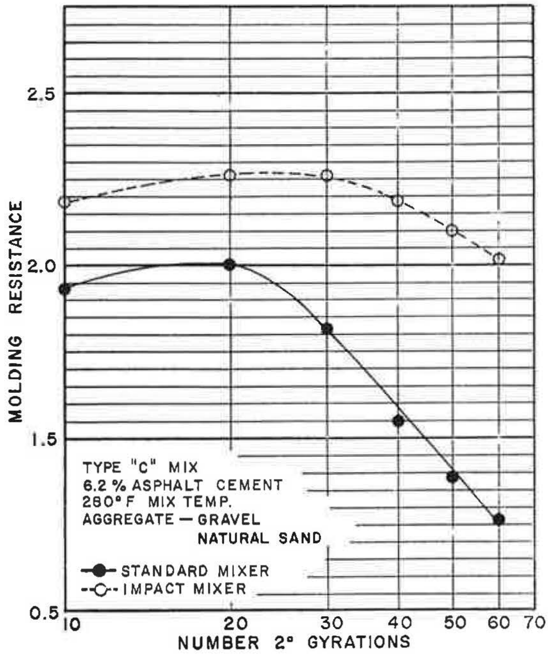

Figure 18.

aggregates. Therefore, it would follow that a strain recording during molding would be more indicative of any difference attributable to the mixing process.

Figures 14 through 16 show the strain recordings for Type A gravel aggregate mixes at three asphalt contents. At the high asphalt content, the resistance to molding drops off rapidly as the mixture increases in density. The impact mix has a resistance to molding very significantly below the standard mix. This resistance to molding must be due to the nature or completeness of the bituminous coating on the individual aggregate particles. It would appear at this high asphalt content that this is too much asphalt for the impact mix. At the normal asphalt content of 5.7 percent, the impact mix has significantly greater resistance to molding than the conventional mix, particularly as the mix is densified. The nature or completeness of the asphalt films on the aggregate particles is perhaps the reason for the measured difference. At the lower asphalt content of 5.3 percent, the conventional mix has a greater resistance to molding than the impact mix. Again, it appears the measured difference is due to the nature of the bituminous films on the aggregate particles. In the conventional mix, at

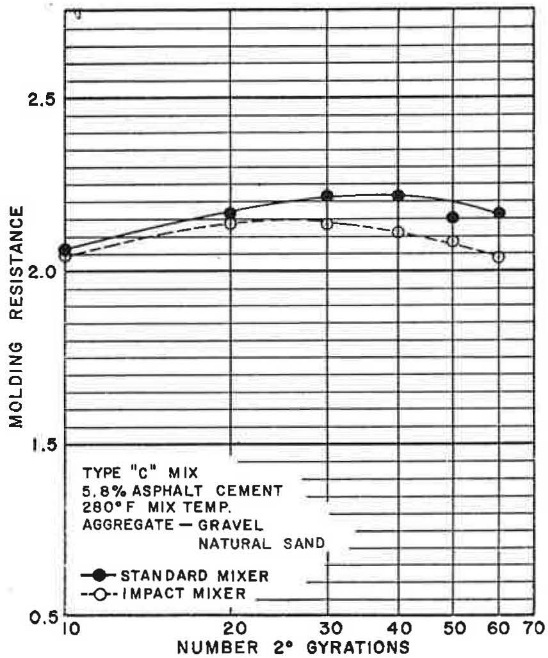

Figure 19.

the low asphalt content, there may be sufficient discontinuity of the bituminous films to allow a part of the particles to have surface contact, thus imparting greater resistance to molding. The resistance to molding for the impact mix at the low asphalt content is very nearly that of the impact mix at the mean asphalt content. Apparently this is due to the nature or condition of the bituminous films and perhaps a quite satisfactory mix can be produced at this lower bitumen content.

Figures 17 through 19 show resistance to molding of the gravel Type C mixes. The same remarks apply to these measurements as those concerning the gravel Type A mixes.

Triaxial Tests

Triaxial test results for the Type C mixes are given in Tables 16 and 17. Figure 20 shows graphically the data in Table 17. The yield stress is plotted against asphalt

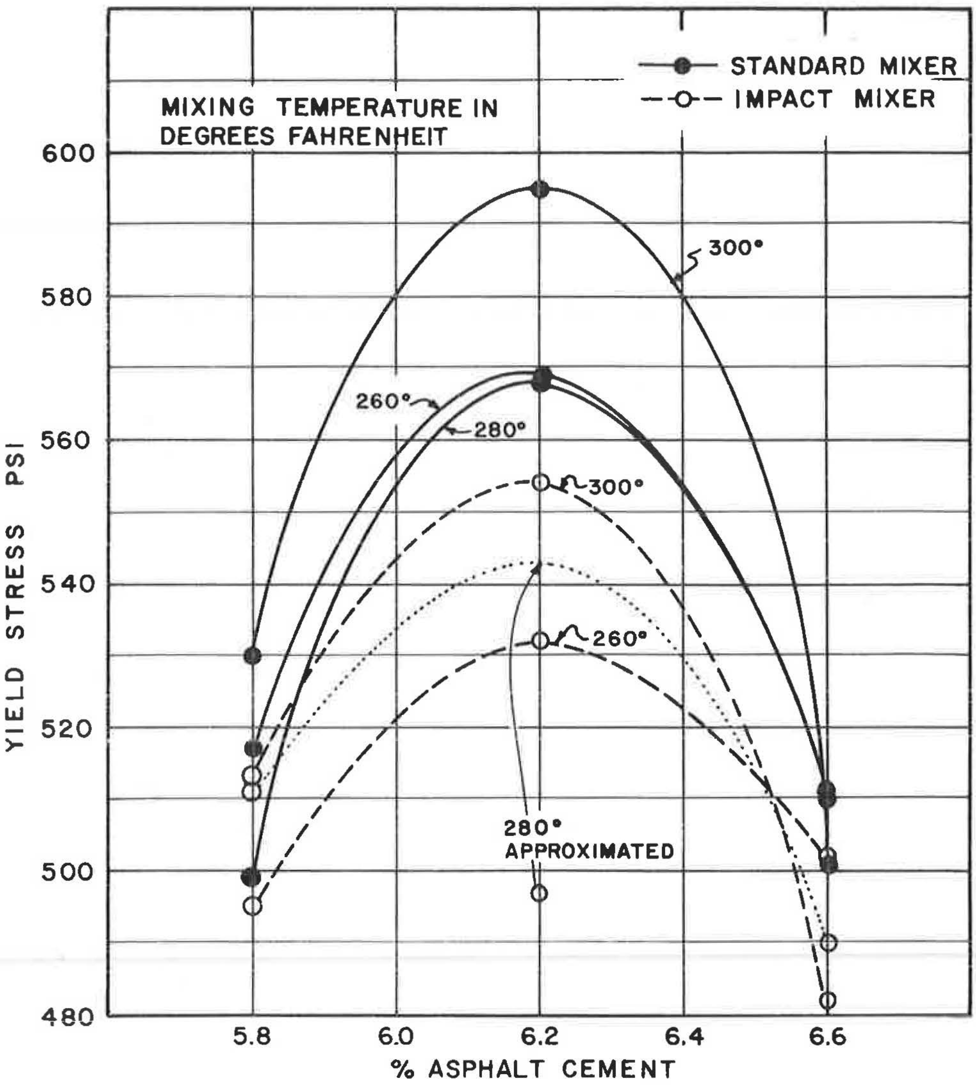

Figure 20. Strength in triaxial compression of Type C mixes, gravel and natural sand aggregate.

content for the mix temperatures, each point representing average data for 3 specimens. The curves indicate the optimum asphalt content for this aggregate combination was at or near the median asphalt content used in the study. In both the impact and standard mixes, it appears the optimum temperature for spreading and finishing the mixtures for maximum strength is about 300 F. As in the case of the unconfined and Marshall specimens, the maximum yield stress or stability was greater for the standard mix specimens.

A stress-strain recording was made for each specimen as it was tested in the triaxial cell. These recordings were continuous through the yield point and to a point where the total load had dropped 50 lb. Some of the specimens required greater degrees of deformation before a point was reached that the total load was reduced to 50 lb under the yield stress. This flatness of the tops of the strain curves is attributed to consolidation of the specimens under axial load, properties of the asphalt films enclosing aggregate particles in the mixtures, a nearly ideal aggregate grading, and particle orientation for the specimen, or a combination of these properties. Figure 20 does illustrate the imperative need in mix design to have the optimum asphalt content for maximum strength of the mixture in the pavement.

In making the triaxial specimens the same molding procedures were used as in the preparation of the unconfined and Marshall specimens. A considerable number of the triaxial specimens were produced before it was discovered that the middle third of each specimen was less dense than the ends. Because the limestone mixtures are more resistant to compaction, triaxial data developed were less consistent than for the gravel mixtures, hence the limestone mixtures were not plotted as in Figure 20.

A study is under way to establish a procedure for the preparation of triaxial specimens in the gyratory compactor in which the density will be uniform for the entire length of the specimens. When this procedure has been established, further study of these mixes produced by both impact and standard methods of mixing will be made.

CONCLUSIONS

1. It is possible, by the impact mixing method, to produce equally satisfactory mixes in a much shorter mixing time. This total mixing time for the complete cycle is perhaps an average of 40 to 50 percent less than for the conventional method. This comparison is made on the basis of the present specification requirement of a 15-sec dry mixing cycle and a 30-sec wet mixing cycle, for conventional mixing. Perhaps in the conventional method the dry cycle could be eliminated and the wet cycle reduced if the mixers were provided with larger paddles and the speed of the shafts increased.
2. The impact method produced a mix having greater workability than the conventional method. This is indicated by the density of the Marshall and unconfined compression specimen and by the density in the completed pavement. Field experience has shown that pavements having a specified minimum degree of density at the time of construction have better service life. The better workability of impact mixes could be a factor directly leading to completed pavements with better service life.
3. The loss of penetration of asphalt used in mixing by both the impact and the conventional methods is approximately the same when all of the conditions of mixing, transportation, placing, and finishing are the same. Under the conditions in this investigation, the loss in penetration of the asphalt as measured by retained percent of the original is approximately 87 percent in mixing, 80 percent in transportation and 73 percent in placing and finishing.
4. The loss in ductility of asphalt used in mixing by both the impact and the conventional methods is the same when all of the conditions of mixing, transportation, placing, and finishing are the same.
5. Impact mixing can result in greater hourly production of asphalt concrete providing certain plant components such as the drier, screens, and bins are increased in size to supply the dried, heated, and graded aggregates.

ACKNOWLEDGMENTS

Grateful acknowledgment is made to Willis Gibboney for his part in this study as Project Engineer on the project and the summation and preparation of data accumulated

from the study. Also to R. R. Litehiser, Chief Engineer of the Ohio State Highway Testing Laboratory and all of those working for him for their fine cooperation and unstinting efforts throughout construction and post project testing.

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Discussion

LADIS H. CSANYI, Bituminous Research Laboratory, Iowa State University.—The Ohio Department of Highways is to be congratulated for undertaking a study of preparing asphaltic concrete mixes by a method other than conventional means used with very little change for more than sixty years. The Department should also be commended for the courage of their convictions in the results of the study in permitting the use of the impact method in their specifications without waiting ten to twenty years for final evaluation. On the basis of the writer's experience with the impact method, their fortitude should be vindicated.

The paper deals primarily with a comparison of asphaltic concrete mixes prepared by the impact and conventional methods. It is also important to note that the impact method can produce asphaltic mixes which cannot be produced by conventional means. Among these is a hot liquid asphalt mastic mix. In Germany, the impact method has been used for a number of years to prepare this mix, known as Gussasphalt, for surfacing, resurfacing and rehabilitating the autobahns and other major streets and roadways.

Although the impact method has not as yet been used for this purpose in the United States, in excess of 40,000 tons of hot liquid asphalt mastic mixes have been prepared by another more recently developed method known as the foamed asphalt process (10, 11) and laid in Dubuque and Ames, Iowa. This mix was used as a thin layer, average ¾ to 1 in. thick, resurfacing for the rehabilitation of old worn portland cement, asphalt and brick pavements (12, 13). These resurfacings have served excellently in excess of three years on streets having grades up to 10 percent and carrying traffic in excess of 10,000 vehicles per day. The cost of this surfacing varied from $0.50 to $0.90 per sq yd, depending on size of contract.

The impact method and the foamed asphalt process can also produce mixes, containing asphalt cement as the binder, that can be stockpiled for months and even years.

Therefore, in evaluating the efficacy of either the impact method or the foamed asphalt process, consideration must also be given to their ability to produce useful products which cannot be produced by conventional means.

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Yardstick for Guidance in Evaluating Quality of Asphalt Cement

PHIL C. DOYLE, Standard Oil Company, Ohio

•THE PENETRATION test, performed at 77 F, 100 g, 5 seconds, has been used for over 40 years on original asphalt cements, on residues from oven loss tests, or on recovered asphalts from hot-mixed pugmill operations to measure the hardening effect of aging on asphalt cement. However, this test cannot be depended on now in many areas due to crossblending of crudes from which the asphalt is produced and to the use of additives and inhibitors.

Observations made by the Standard Oil Company of Ohio Laboratory on a special test road built in 1953 on the State highway system near Millersburg, Ohio, showed the inadequacy of the penetration test as a quality guide when the penetration test was run on recovered asphalts. Asphalt cements of the 85-100 penetration grade from different refineries and different crudes were used following the same mixing and same laying procedure with significant differences as given for two of the asphalts in Table 1. The recovered penetrations were approximately the same yet one pavement cracked extensively, while the other remained unaffected. The recovered asphalt when tested for ductile characteristic at 55 F, 1 cm per min clearly identified the difference in the two asphalts.

Other observations pointed out that ductilities on original asphalts run at 77 F, 5 cm per min did not give a definite indication of the sensitivity to hardening of an asphalt cement in the hot-mixing operation. It was observed that there were asphalts which would have ductilities of 100+ cm on the original asphalt at 77 F, 5 cm per min, but which would still manifest early cracking when incorporated in a properly prepared and properly placed and compacted pavement. These asphalts showed a noticeable difference when tested for ductility as supplied at a lower temperature as given in Table 2.

For years various publications (Appendix C) have pointed towards the use of low temperature ductility as a criterion for spotting early cracking asphalt cements not particularly on the original asphalt but more importantly the tests run on the asphalt recovered from cores from the pavement. Paving engineers have observed that cracking occurs as asphalt pavements gradually chill down to lower temperatures in passing from the summer months into the winter period.

Asphalt cements were laboratory tested for expansion and contraction at different temperatures using the linear thermal expansion of penetration grade asphalt test (see Appendix A). Curves (Fig. 1) plotted from these tests reveal definite breaking off or transition points at which the asphalt changes from a plastic to a solid form. These points for asphalt cements were between temperatures of 30 F and 50 F. This same method of testing on asphaltic concrete indicated that transition points were close to 45 F for typical mixtures.

Because it was found by the laboratory that 77 F was too warm a temperature to run ductilities having any significance on original or recovered asphalts, a series of tests were run on both original asphalts and recovered asphalts at 55 F, 1 cm per min. The results are shown in Table 3. All five samples showed a 100+ cm ductility at 77 F, 5 cm per min on the original asphalt, whereas there was quite a difference in the ductility of the samples when run at 55 F, 1 cm per min. The ductilities on the recovered asphalts pointed up even more noticeable differences.

The results of these tests at 55 F, 1 cm per min and others were still not capable of equipping one to separate the good from the bad accurately. Testing ductility at 45 F, 1 cm per min was inaugurated and finally adopted as being a more desirable temperature and speed to evaluate ductile characteristics on both original and recovered asphalt cements.

Paper sponsored by Committee on Relation of Physical Characteristics of Bituminous Mixtures to Performance of Bituminous Pavements.

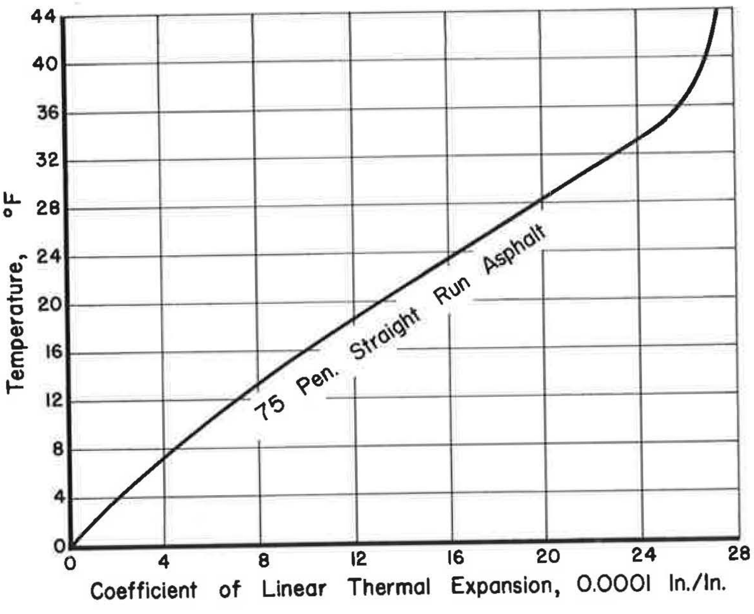

Figure 1.

TABLE 1

ASPHALTS USED IN TEST ROAD 2, 1953

Penetration and Ductility	Asphalt A Cracked After 2 Yr	Asphalt E Not Cracked After 3 Years
Orig. asphalt:		
Pen. at 77 F, 100 g, 5 sec	94	88
Hot-mix plant simulating test recovered asphalt:		
Pen. at 77 F, 100 g, 5 sec	49	48
Duct. at 55 F, 1 cm per min	7	16
Asphalt recovered from cores taken after 3 years:		
Duct. at 55 F, 1 cm per min	6	12

TABLE 2

ORIGINAL ASPHALTS

Test	Asphalt A	Asphalt E
Penetration at 77 F, 100 g, 5 sec	94	88
Ductility at 77 F, 5 cm per min	100+	100+
Ductility at 45 F, 1 cm per min	9	28

TABLE 3

ASPHALT CEMENTS PRODUCED FROM VARIOUS CRUDES, 1958

Asphalt	Original Asphalt			Hot-Mix Plant Simulating Test Run 325 F[1]	
	Pen. at 77 F, 100 g, 5 sec	Duct. at 77 F, 5 cm	Duct. at 55 F, 5 cm	Pen. at 77 F, 100 g, 5 sec	Duct. at 55 F, 1 cm
Crude 1	50	100+	100+	36	16
Crude 2	53	100+	82	43	20
Crude 3	53	100+	28	35	7
Crude 4	54	100+	54	39	10
Crude 5	55	100+	100+	39	19

[1]Recovered asphalt.

TABLE 4

ASPHALT CEMENTS FROM VARIOUS CRUDES, 1958

Crude	Original Asphalt		Hot-Mix Plant Simulating Test[1]	
	Pen. at 77 F, 100 g, 5 sec	Duct. at 55 F, 1 cm	Duct. at 55 F, 1 cm	Pen. at 77 F, 100 g, 5 sec
A	86	100+	30	37
B	89	100+	26	36
C	88	100+	30	40
D	88	100+	55	48
E	91	100+	14	61
F	87	100+	46	57

[1]Recovered asphalt.

TABLE 5

ASPHALT RECOVERED FROM TEST ROAD[1] BUILT IN 1953

Core	Duct. at 55 F, 1 cm	Pen. at 77 F, 100 g, 5 sec
1	5	33
2	5	31
3	8	48
4	8	39
5	6	41

[1] Cracked extensively after three years of service; all cores from same road.

TABLE 6

LOSS OF DUCTILE[1] CHARACTERISTICS

Asphalt	Pen. Range	Pen. Avg.	Oven Weath. Temp. (°F)	Ductility
Str. asph.[2]	85-100	88	-	150+
	70- 85	78	-	130
	60- 70	63	-	43
	50- 60	56	-	39
Mixed and oven weath.[3]	85-100	-	289	19
	70- 85	-	293	13
	60- 70	-	298	8¼
	50- 60	-	302	7½
Mixed and oven weath.[4]	85-100	-	325	10½
	70- 85	-	325	6½
	60- 70	-	325	5¼
	50- 60	-	325	5

[1] Ductility measured at 45 F, 1 cm per min, June 1962.
[2] As delivered to contractor.
[3] Hot-mix plant simulating test; correct temperature for each grade.
[4] Hot-mix plant simulating test; 325 F for all grades.

TABLE 7

DUCTILITY[1] OF ASPHALT CEMENT RECOVERED FROM CORES FROM HIGHWAYS

Location	Age (yr)	Pen. at 77 F, 100 g, 5 sec	Ductility at 45 F, 1 cm	Condition	Air Voids (%)	Asphalt (%)
Fairfield (W. 14-Scranton) Cleveland	14	17	0	Cracked	5.5	6.8
Memphis (Pearl-W. 45) Cleveland	13	41	4¼	Cracked	4.0	7.0
Baltic (W. Blvd.-W. 95) Cleveland	13	28	4	Cracked	3.7	6.7
Maine Turnpike[2] (cored 1961)	14	21	3⅜	Cracked	0.3	6.5
Fairfield (Professor-W. 14) Cleveland	14	78	15½	No Cracks	3.0	6.8
Memphis (W. 45-Border) Cleveland	12	63	9½	No Cracks	1.6	5.8
Clifton (W. 95-Lake Road) Cleveland	13	29	14	No Cracks	4.6	7.8

[1]At 45 F, 1 cm per min.
[2]From different refinery and different crude.

TABLE 8

AVERAGE OBSERVATIONS HOT-PLANT MIXES, ASPHALTIC CONCRETE MIXED AT CORRECT TEMPERATURE

Penetration	Asphalt	
	Orig. Duct.	Recovered Duct.[1]
85-100	150+	15-25
70- 85	115	10-14
60- 70	35	6- 9
50- 60	15+	5- 8

[1] At 45 F, 1 cm.

The asphalt from Crude 3 was the only cement which field tests indicated showed signs of some early deterioration. If these recovered asphalts had been run at 45 F, 1 cm per min, the deficiency would have been much more pronounced.

A test somewhat similar to the widely-known Shattuck test which attempts to simulate the action taking place in a hot-mix pugmill operation in the production of asphaltic concrete mixtures has been standardized. It is identified as the hot-mix plant simulating test (see Appendix B). Its procedure combines an adaptation of the BPR mixing and weathering test with the Abson recovery test.

Samples of asphalt cement produced from different crudes were tested both on the original asphalt and the recovered asphalt from the hot mix plant simulating test. Results are given in Table 4.

Table 5 includes tests on recovered asphalts from a test road in Indiana. This road cracked extensively and the low ductility recordings would identify the lack of stretchability at relatively low temperatures. Had these recovered ductilities been run at 45 F, 1 cm per min, they would have been more clearly defined. Original asphalt was 60-70 penetration at 77 F, 100 g, 5 sec. Tables 4 and 5 were completed prior to adoption of the 45 F, 1 cm per min test condition.

Low ductility in a pavement can be caused by overheating of the asphalt in the pugmill. Table 6 shows the tremendous drop in ductile characteristic which an asphalt experiences when subjected to hot mixing in a pugmill.

Table 6 gives results from four different grades of asphalt cement when mixed at the correct temperature. (Proper mixing temperature for any given asphalt can be determined by the procedure outlined in Appendix B.) These cements were produced from the same crude. The mixing and weathering test simulates mixing in an actual pugmill operation. Extracted asphalt will show ductility at 45 F, 1 cm per min, close to that retained in the highway. If this test reveals an 18-cm ductility at 45 F, 1 cm per min, it is reasonable to expect asphalt mix from a commercial plant to run 25 cm. This difference is due to a specified asphalt content of 4 percent, whereas actual plant mixes will normally contain a higher asphalt content.

It is easy to see why paving engineers pick the softest asphalt consistent with stability to use in mixes. A great deal of work has been performed in an attempt to inhibit this loss of ductile characteristic in the hot-mixing operation. There are some inhibitors and additives available which will contribute materially toward preventing the tremendous drop in ductility.

These data emphasize the desirability of testing asphalt extracted from paving-type mixtures prior to using the asphalt cement in hot-mix construction. The results on such mixes will give a very good indication of what one can expect of the performance of the pavement in time.

Table 7 gives the results of asphalt cement recovered from cores taken from various highways. Pavements that had above 8-cm ductility at 45 F, 1 cm per min after various periods of service were not cracked and not raveled. The pavements showing less than 8-cm ductility at 45 F, 1 cm per min were in each case cracked or raveled.

Samples of paving mixtures produced at various plants and using many different asphalt cements from different crudes and refineries were tested by the laboratory. The test results were averaged and are given in Table 8, indicating that where the softest asphalt cement (85-100 penetration) was used, the original ductility was 150+ cm but recovered ductilities were between 15 cm and 25 cm. All other grades also lost most of their ductile characteristic in the mixing operation.

SUMMARY

Low temperature ductility provides a method of measuring the future service behavior of asphalts and the pavements in which they are incorporated. An improved laboratory method provides a tool whereby the simulation of the hot-mix pugmill operation on a laboratory basis can be duplicated. This procedure will allow an evaluation of the asphalt before it is placed.

It has been determined that any paving mixture which contains an asphalt which shows a ductility exceeding 8 cm should be free from cracking. A value somewhat higher than 8 should be established to provide a safety factor. It is suggested that a minimum ductility at 45 F, 1 cm per min run on extracted asphalt from the paving mixture should be in excess of 25 cm at the start of its service record. This 25-cm minimum ductility correlates with the 18-cm minimum ductility in the hot-mix plant simulating test (Appendix B). The explanation of the difference between 18 cm and 25 cm lies in this test's requiring a specified amount of asphalt (4%) whereas field plants vary upward in their percent of cement used.

Other work in connection with this ductility study has brought out other factors which may contribute to the hardening of the binder in an asphalt pavement:

1. A correct mixing temperature in the pugmill for the particular asphalt cement being used should be adopted. If the correct temperature was exceeded, additional loss of ductility was noted.
2. An asphalt content as high as possible, consistent with maintaining a desired level of stability, should be used. Thin asphalt films should be avoided.
3. The paving mixture should be compacted at the time of construction to show less than 5½ percent voids. Equipment permitting the reading of density tests made on the surface of the pavement immediately following the roller would be most helpful.

Appendix A

LINEAR THERMAL EXPANSION OF PENETRATION GRADE ASPHALT

The purpose of the test is to determine accurately the linear movement of a briquette of asphalt when subjected to controlled variations in temperature.

General Description

A small briquette of solid asphalt is supported in a methanol-water bath. As the temperature is changed, measurements of sample movement are observed with a cathetometer.

Sample Preparation

The specimen is a casting of solid asphalt molded into a 1½- × 1½- × 12- in. briquette, using a demountable brass mold. The mold is coated with a thin film of Dow Corning 200 fluid before casting to minimize sticking. Small threaded Invar metal plugs are carefully embedded into the vertical face of each end of the specimen during the casting. One is used to anchor the specimen rigidly to an Invar frame during the test; the other, to secure a reference point used for cathetometer observations. The underside contact surface of the sample has a thin rubber membrane affixed to it to eliminate the possibility of sample adhesion to the specimen holder, particularly at higher temperatures.

Instrumentation

Elongation measurements are made by following the linear movement of an index point embedded in the free end of the sample at the time it is cast. This movement is measured with a rigidly-mounted cathetometer.

Temperature measurements are made with thermometers embedded in a second briquette of asphalt, also supported in the bath. This second briquette has the same cross-section dimensions as the test specimen. Temperature ranges from -10 to +80 F have been used in this test.

Bath Coolant

The heat transfer medium consists of a methanol-water solution adjusted so that the gravity of the combination is less than the asphalt specimen to minimize its buoyant effect on the specimen.

Procedure

The asphalt specimen is supported in an Invar metal frame with one end of the specimen free and the other anchored to the frame by means of a small threaded Invar plug. The specimen and frame are immersed in an insulated methanol-water bath. The coolant is circulated in this table top bath from an adjoining bath equipped with a coolant pump and provisions for varying temperatures of the heat transfer media under carefully controlled conditions.

Measurements of linear sample movements are made in $\pm$ 10 F units. The temperature of the bath is changed 10 F, the specimen conditioned for one hour and a linear measurement made. The temperature is again changed 10 F, the sample held at this new temperature for one hour and a linear measurement again recorded. This sequence with a series either of increasing or decreasing temperatures is followed until sufficient observations are made to record the linear thermal characteristics of the asphalt specimen adequately.

Transition Point

When a sample of asphalt is subjected to slight increases in temperature a definite linear expansion is observed. Above a certain temperature this movement ceases and it is assumed that at this point the specimen has changed from a rigid material to a plastic, amorphous substance. This characteristic and observation is relative and dependent on the test conditions used, but it forms an interesting comparison of asphalt behavior when numerous samples are checked under similar conditions.

Appendix B

HOT-MIX PLANT SIMULATING TEST

This is a method for determining the sensitivity of an asphalt to mixing and heating at a certain film thickness, measured by ductility at a relatively low temperature and testing speed.

Oven Weathering

The temperature of the asphalt is measured with a Saybolt-Furol viscometer at 120 sec (Fig. 2). The measured temperature becomes the mixing temperature of the asphalt and sand and also the oven temperature in which the mixture is placed for one-half hour after mixing. This temperature, usually falling between 290-310 F for normal 85-100 penetration asphalts, is specified to the nearest degree to insure equal film thickness for all asphalts when mixed with the sand. Plotting of viscosity and temperature on semilog paper aids in finding the 120-sec viscosity temperature.

Figure 2.

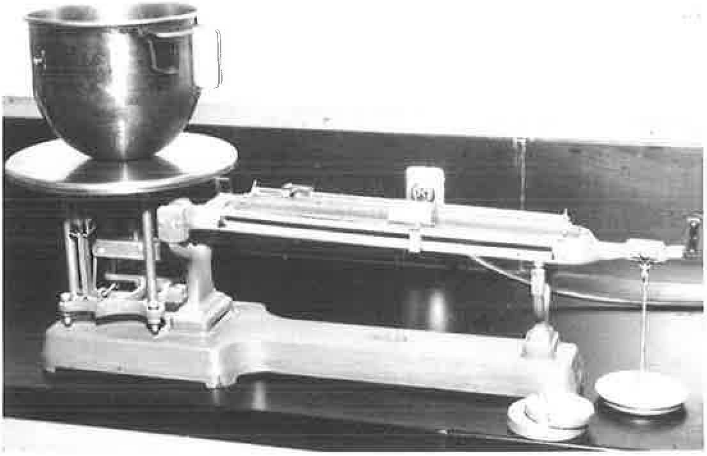

Figure 3.

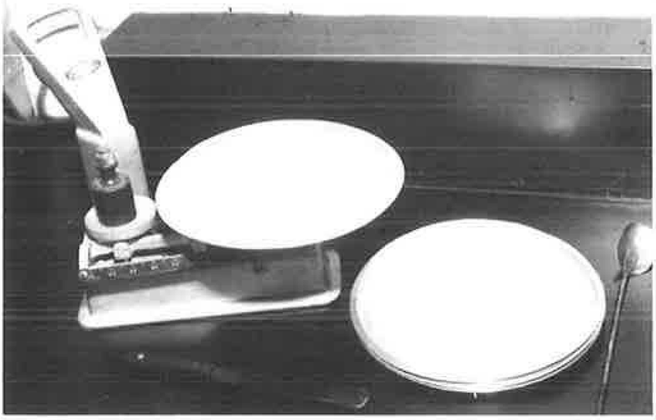

Figure 4.

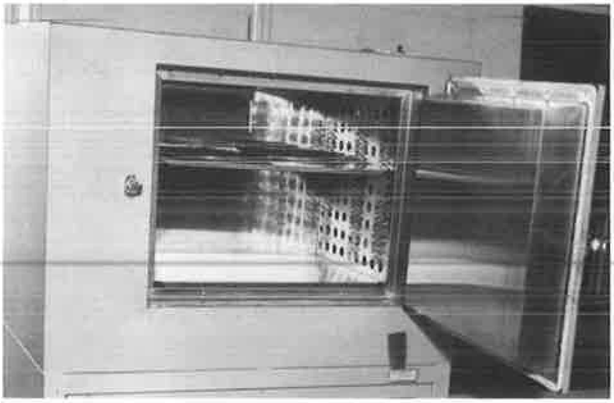

Figure 5.

Figure 6.

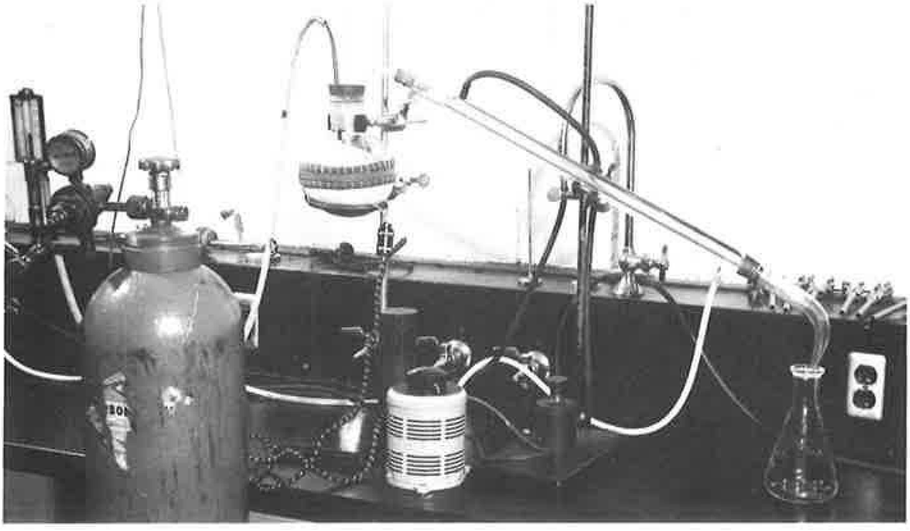

Figure 7.

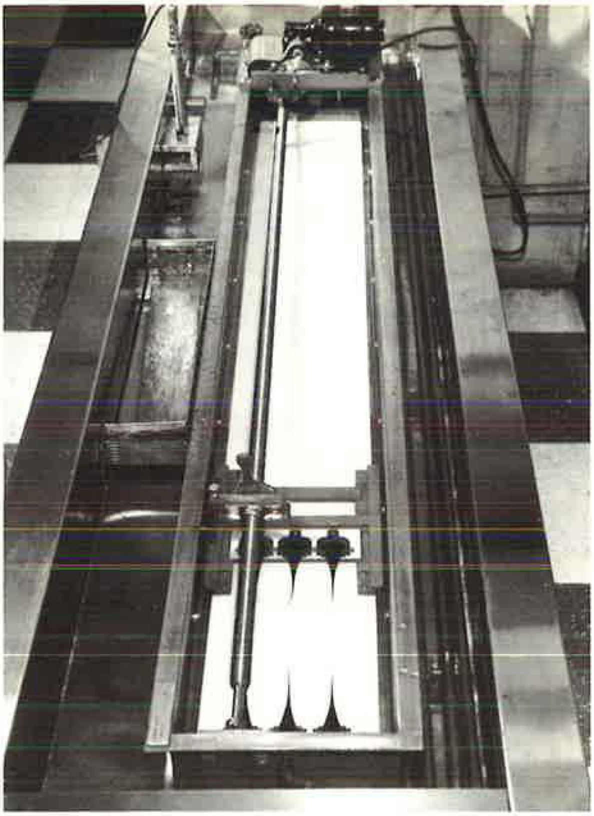

Figure 8.

Figure 9.

Exactly 2, 000 g of Ottawa sand (ASTM C-190, 20-30 sieve) is placed in a bowl (Fig. 3). Approximately 160 g of asphalt is placed in a clean container. The sand and asphalt are heated separately in an oven to the 120- sec viscosity temperature. When the asphalt and sand have reached temperature, the previously tared bowl and sand are removed from the oven and placed on the balance. Exactly 80 g of asphalt is placed in the bowl with the sand.

The asphalt and sand are mixed immediately in the Hobart mixer by the preheated, rigid paddle for exactly 2 min. The main shaft speed is set at 60 rpm and causes the paddle speed to be slightly over twice the 60-rpm rate. The heat is maintained within $\pm$ 10 F with three 250-w infrared lamps (see Fig. 11). After a 2-min mixing, the bowl with the hot mixture is removed and the hot mixture is then batched into 2 aluminum pizza pans (Fig. 4). Exactly 900 g of mixture is placed into each pan and smoothed out uniformly so that the mixture covers the inside surface evenly in area and in depth. The pans are No. 814 gage, 13-in. O.D., 12-in. I.D. and $^3/_8$-in. deep throughout the I. D. The pans with the mix are placed in a safe place resting on a table and are cooled to room temperature for 1 hour, minimum.

After the mixture has reached room temperature, it is ready for weathering in the oven (Blue-M POM 120, oven accuracy $\pm$ $^1/_2$ C, 25- $\times$ 20- $\times$ 20- in. I.D.; forced convection 50 fpm). The mixture in pans is placed on a shelf in the oven (Fig. 5) for exactly $^1/_2$ hr at the exact temperature which gives a 120-sec viscosity. The $^1/_2$-hr weathering is measured immediately from the time the pans are introduced into the oven. Care should be taken to achieve the 120-sec temperature as quickly as possible. Technique will achieve this temperature within a minute.

After $^1/_2$-hr baking, the pans are removed from the oven and allowed to cool to room temperature for at least 1 hr before recovery. The pans are placed on a flat surface while cooling. (After the oven heating, the sample may be allowed to stand overnight before extracting with benzene.)

The mixture is scraped from the pans into the Dulin Rotarex (Fig. 6) with a putty knife and the benzene added, allowing 250 ml of benzene (C.P.) for each panfull. After soaking about 10 min, the solution is rotated off. The solutions from several identical samples may be combined before distillation.

Asphalt Recovery Procedure

The asphalt recovery procedure is begun with centrifuging the solution. A sample is heated to 250 to 270 F temperature. The mix is broken up with a spoon or spatula and 1, 000 g is introduced into the previously tared Dulin Rotarex bowl. Approximately 250 cc of benzene, C.P. grade, is added and the mixture allowed to soak 20 min. The mixture is then rotated and the benzene-asphalt solution extracted. The remaining aggregate and asphalt may be washed again with benzene (C.P.) and allowed to soak 20 min, then rotated off. The two solutions are then combined.

The solution is placed into two 250-ml centrifuge bottles and centrifuged at 870 g's for 45 min to remove all fines (see Fig. 12). In the hot-mix simulating test this time can be reduced to 15 min because of the small amount of fines present in the Ottawa sand. The solution is decanted from the bottles into a 1, 000-cc spherical distilling flask.

The distilling flask (see Fig. 9) has a side arm 77 mm below the top of the neck and is at a 75^0 angle (see Fig. 13). The flask is heated by two Glas-Col Series O heating mantles controlled by two Variacs. An ASTM 30-180 F thermometer is inserted in the flask and positioned so that the bulb is $^1/_4$ in. from the bottom. The side arm of the flask is connected to a straight innertube condenser.

When approximately 200 ml of benzene has distilled over, heating is suspended and the residual solution is transferred to a 500-ml round bottom, short neck, 50/50 joint flask (Fig. 7), heated by a Glas-Col heating mantle, 500-cc capacity and controlled by one Variac. The flask is stoppered by one size 11 neoprene stopper. Passing through the stopper is a $^3/_{16}$-in. copper tube for the introduction of CO_2 (see Fig. 14). It is important that the distributing holes in the tube foot be directed towards the flask bottom. Also passing through the stopper is an ASTM 85-392 F thermometer, whose

Figure 10.

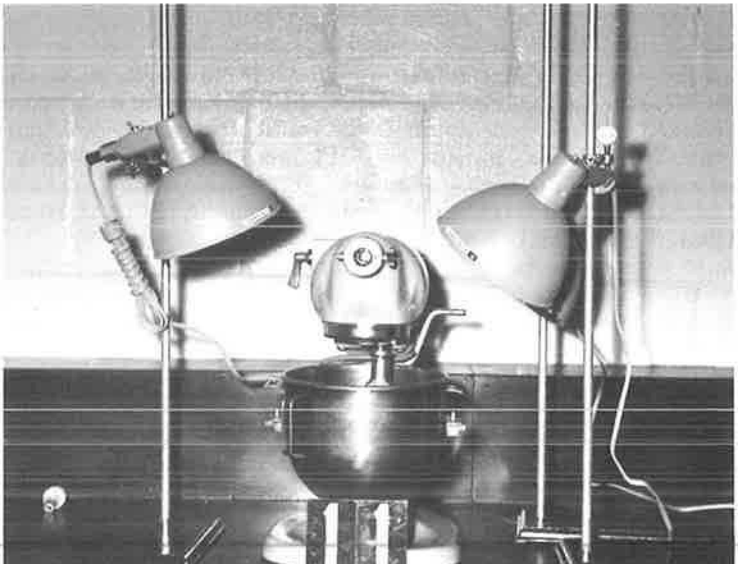

Figure 11.

Figure 12.

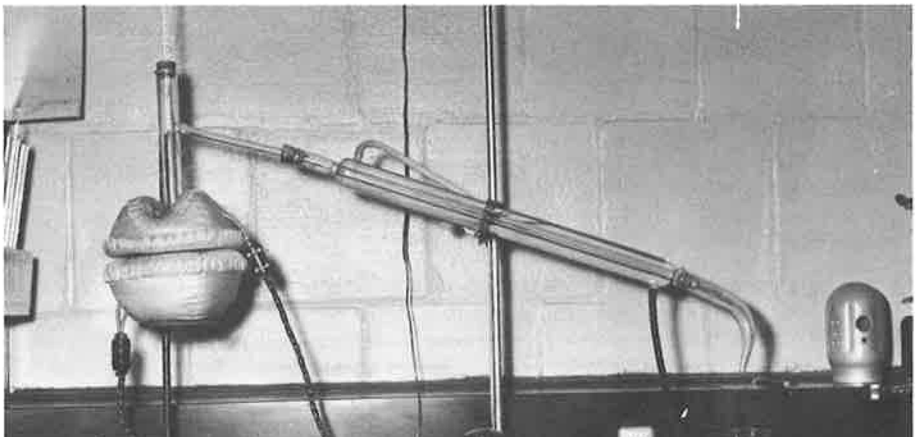

Figure 13.

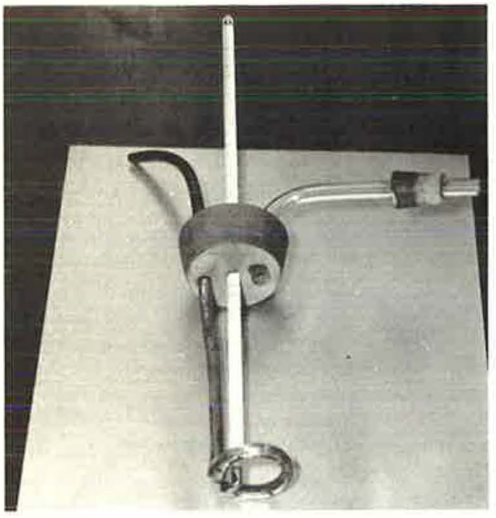

Figure 14.

bulb is centered in the middle of the copper tube foot, 1/4 in. from the flask bottom, and a glass tube for the delivery of benzene vapor to the condenser.

Heat is applied to the flask and the benzene distilled off at the rate of 50-70 drops per minute. When a temperature of 300 F is reached, CO_2 is introduced slowly until 325 F ± 5 F is reached, at which point a rate of 875 cc of CO_2 per min, ± 100 cc, should be obtained. During the CO_2 introduction, it will be necessary to turn the Variac down or possibly off for a short period to avoid overheating. When the drops from the condenser are 30 sec apart the end point of distillation is 15 min additional at 325 F.

The heat is discontinued and the apparatus disassembled. The CO_2 is allowed to run to prevent the copper tubing from clogging. The tube assembly is placed in solvent for cleaning. The asphalt is poured into a 3-oz penetration tin and stirred 1 min to remove the CO_2 bubbles. In no case should the asphalt be permitted to remain in solution more than 10 hr.

Ash

A sample of the asphalt is ashed in accordance with AASHO Method T-111-42. The ash content should be below 0.5 percent. In no case should it be above 1 percent by weight. If a sample yields an ash higher than 1 percent, extract a new sample of asphaltic concrete and increase the centrifuge time accordingly.

Ductility Tests

The recovered asphalt is tested for ductility according to ASTM Designation: D-113-44 at a temperature of 7.2 C ± 0.1 C (45 F ± 0.2 F) and at a rate of pull of 1 cm/min (Fig. 8). In preparing the sample the asphalt is heated in a penetration cup to 300 F with stirring. It is immediately placed in a vacuum bell jar and subjected to a vacuum of 10- 15 mm Hg until bubbling has ceased. It is then removed and poured. In some cases additional heating may be necessary before pouring. The molds are cut off with a sharpened, stiff back putty knife.

Sampling Technique

When obtaining pavement samples as with a coring machine (Fig. 10), cores should be taken in the wheel tracks for maximum sample density. In this area the optimum weather resisting characteristics occur, due to a lowering in the voids content of the mix. Sampling between lanes or at the edge is suspect due to spillage of oil and grease.

Appendix C

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Variability in the Testing and Production Of Bituminous Mixtures

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•LIKE most production processes, the production of bituminous mixtures is subject to variation. This variation can broadly be attributed to two major sources related to (a) mixing, composition and the characteristics of the constituents, and (b) sampling and testing.

From a practical standpoint, a statistical evaluation of the sources and magnitude of the variability is almost essential to:

1. Establish the capability of a given testing procedure.
2. Establish the capability of the production process with respect to each property.
3. Refine the production process where necessary and practicable.
4. Set realistic specifications.
5. Provide statistical measures to control the process and evaluate experimental data.

Figure 1 shows how these two major sources can be broken down into components. The supplier-to-supplier variation is related to factors such as uniformity of composition, the characteristics of the materials, and the efficiency of the process. Different plants have different process capabilities, which are reflected in the following:

1. Components related to mixing, composition and characteristics of the constituents:
   (a) Day-to-day variation in the process efficiency and the uniformity of the mixture.
   (b) Batch-to-batch variation in uniformity of the material.
2. Components related mainly to sampling and testing:
   (a) Sample-to-sample variation of the uniformity of the material within a batch and the representativeness of the samples.
   (b) Laboratory-to-laboratory variation.
   (c) Operator-to-operator variation.
   (d) Day-to-day variation due to the changes in testing conditions on different days.
   (e) Briquet-to-briquet variation, expected between determinations performed by a single operator on the same sample using the same apparatus on the same day.

Odasz and Nafus (8) and Shook (9) studied plant-mix variation, whereas Corbett (12) and Vokac (11) studied laboratory mixtures. The scope of these four investigations is compared in Table 1, from which it is evident that they differ in many respects. The statistical measure of variation quoted in each case reflects these differences (materials, mixing procedures, etc.) and also differences in the basic variation components, which were combined in the estimate. This latter difference arises partly as a result of the method of statistical analysis, given in the last column of Table 1.

The complexity of the variation problem and the importance of clear definitions for the scope of any statistical studies are apparent from the foregoing.

The purpose of this paper is to illustrate by practical examples further applications

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Paper sponsored by Bituminous Division, Department of Materials and Construction.

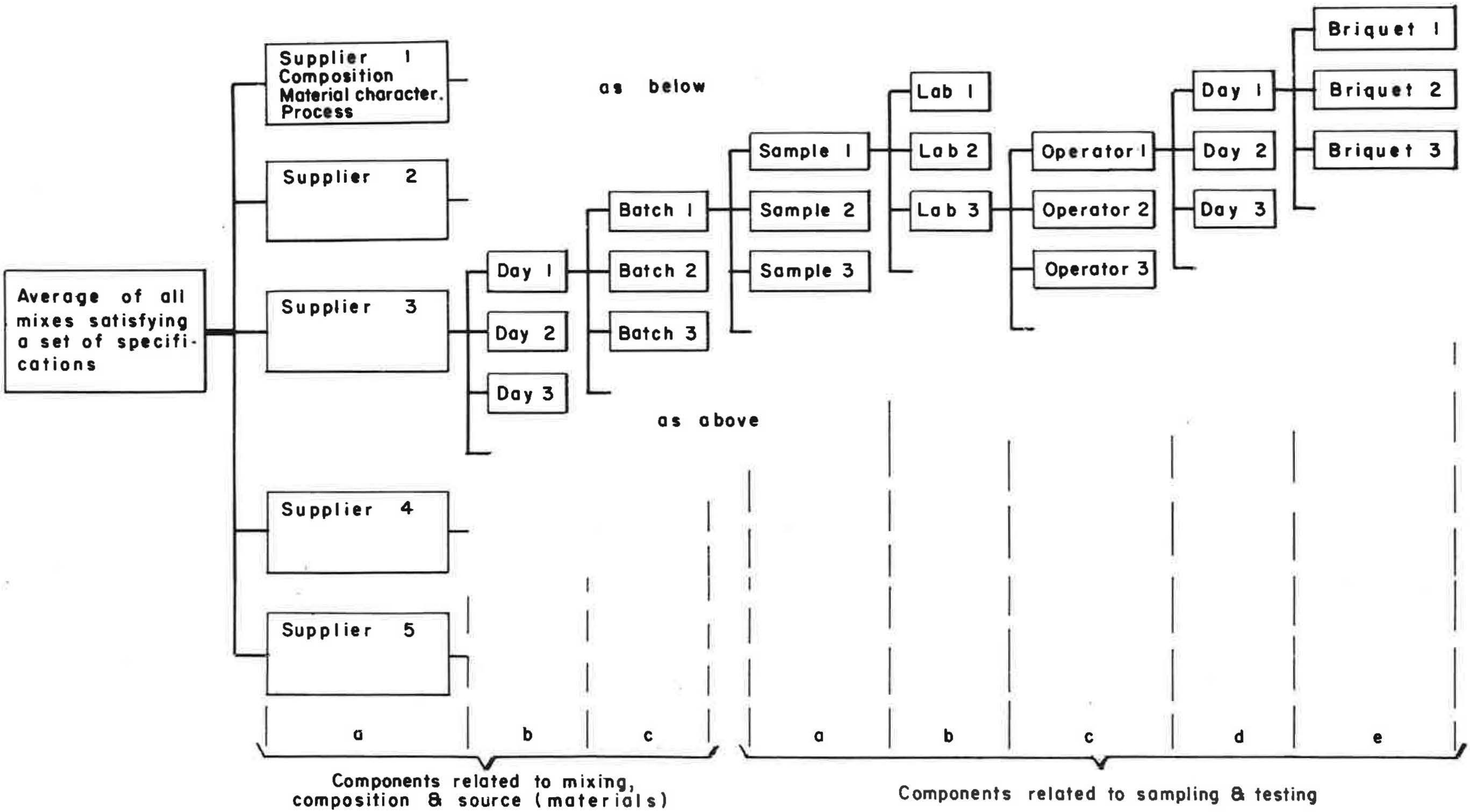


Figure 1. General sources of variation.

TABLE 1

LIMITATION AND SCOPE OF FOUR VARIATION STUDIES

Reference	Type of Materials	Size Distribution	Description of Sample and Test	Component of Variance[1]
Odasz and Nafus (8)	Plant mix: 100-120 pen. asphalt; crushed limestone agg.; natural sand.	1-in. max., 57% pass No. 8, 15% pass No. 200	Two samples obtained from separate batches in one truck; hand compaction, 50 blows per face; one briquet per sample; one briquet per run.	$\sqrt{\sigma^2_{BB} + \sigma^2_{res}}$ (plant)
Corbett (12)	Lab. mix: 85-100 pen. asphalt; (a) crushed stone screening, blank sand; (b) slag, screening, blank sand	½-in. max., 43% pass No. 10, 5% pass No. 200	New batch for each briquet; hand compaction, 50 blows per face; one briquet per run.	$\sqrt{\sigma^2_{BB} + \sigma^2_{res}}$ (plant)
Shook (9)	Plant mix: 85-100 pen. asphalt, crushed dolomitic limestone coarse agg.; natural coarse and fine sand; mineral filler	1-in. max., 46% pass No. 10, 5.2% pass No. 200	Two samples obtained from separate batches in one truck; mechanical compaction, 50 blows per face; two briquets per run.	$\sqrt{\sigma^2_{BB} + \sigma^2_{res}/2}$, (plant)
Vokac (11)	Lab. mix; 85-100 pen. asphalt; gravel; natural sand; Ottawa sand; limestone filler	⅜-in. max., 54% pass No. 8, 6% pass No. 200	Factorial design; 4 samples per day, 4 days per week, 4 consecutive weeks; new batch for each briquet; kneading compactor; 2 samples per run.	σ_{res}

[1]BB = between batches; res = residual.

of statistical methods. The study covers a laboratory and a field investigation of two mixtures (surface and base) produced by two plants. The study is reported in the following four parts:

1. An analysis of the repeatability of the Marshall stability and density tests and Rice's maximum density test. The limitations of the Marshall test and Rice's test were determined by analysis of variance studies made on test results obtained from laboratory-prepared mixtures. The scope and limitations of the experiment are shown in Figure 2.
2. An analysis of the variations occurring within a well-controlled production process. The extent of production variation was assessed by the statistical analysis of test results from controlled production samples. The scope of this work is shown in Figure 3.
3. A discussion of the influence of unavoidable process variations on mix design and the setting of specifications.
4. A discussion of the use of statistical control charts.

No attempt was made to study the technical validity of the Marshall test in assessing the in situ quality of the material.

REPEATABILITY OF MARSHALL AND RICE'S MAXIMUM DENSITY TESTS

Strictly speaking, testing variation or precision is the variability in results which can occur between replicate tests made on the same test piece or specimen over a number of days using a variety of operators and testing machines. However, in the case of the Marshall test, because the test is destructive, a different briquet must be used each time and a certain amount of unmeasurable and unremovable variability due to sampling and mixing (lack of homogeneity) is introduced.

Because of limitations on the number of operators and testing machines available for this study, the work was necessarily restricted to only one operator and one set of compaction and testing equipment. The influence of different operators and test equipment must not be neglected, however, where interlaboratory comparisons are being made, and these can be assessed through round robin tests (1).

This and other studies (2) have shown that, for a given equipment-operator combination, the magnitude of the "testing" variation of the Marshall tests may vary with each "design formula-supplier" combination. This is demonstrated in Table 2, which gives the "between duplicate briquet" variability for material supplied by two contractors to

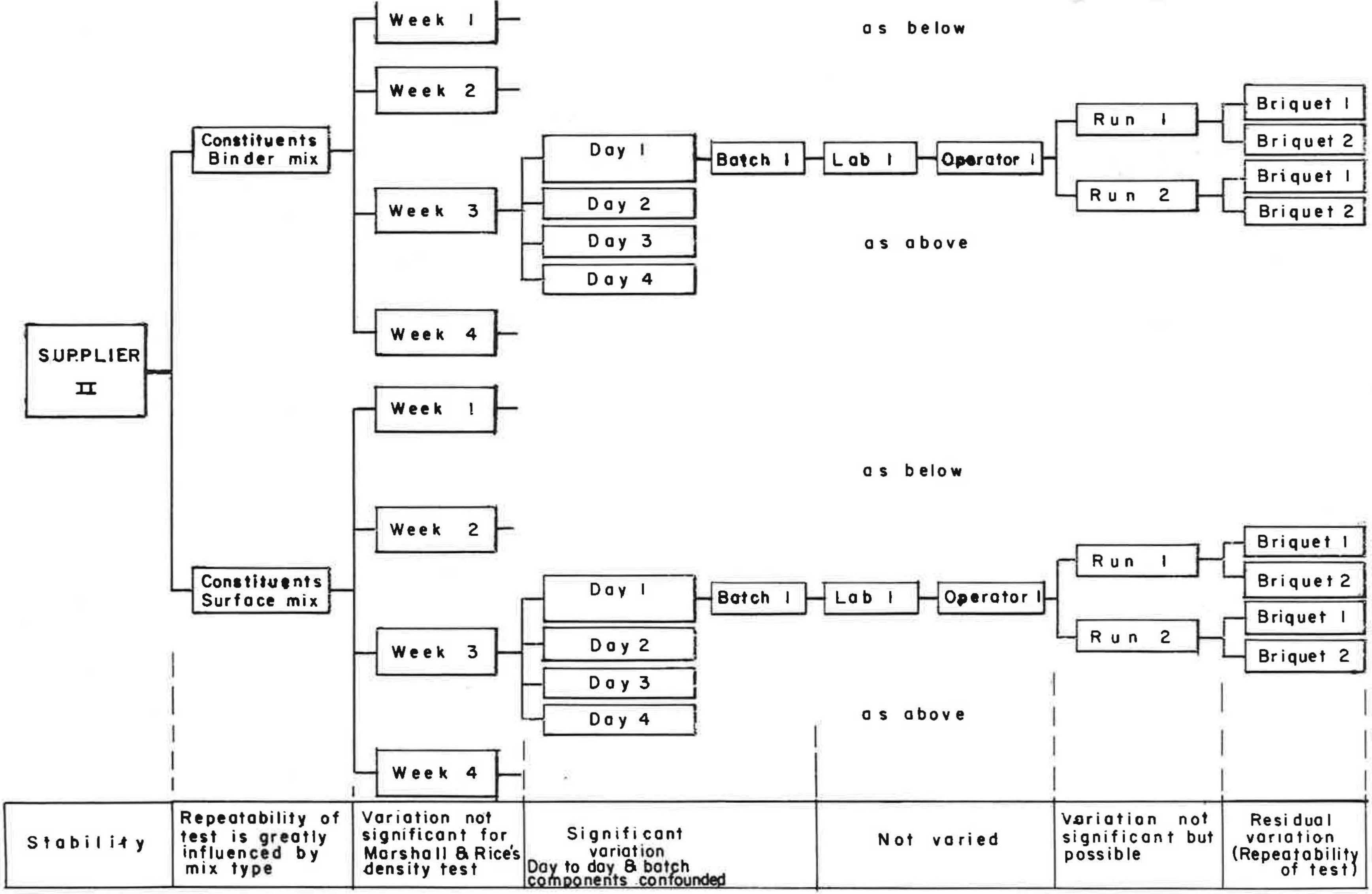
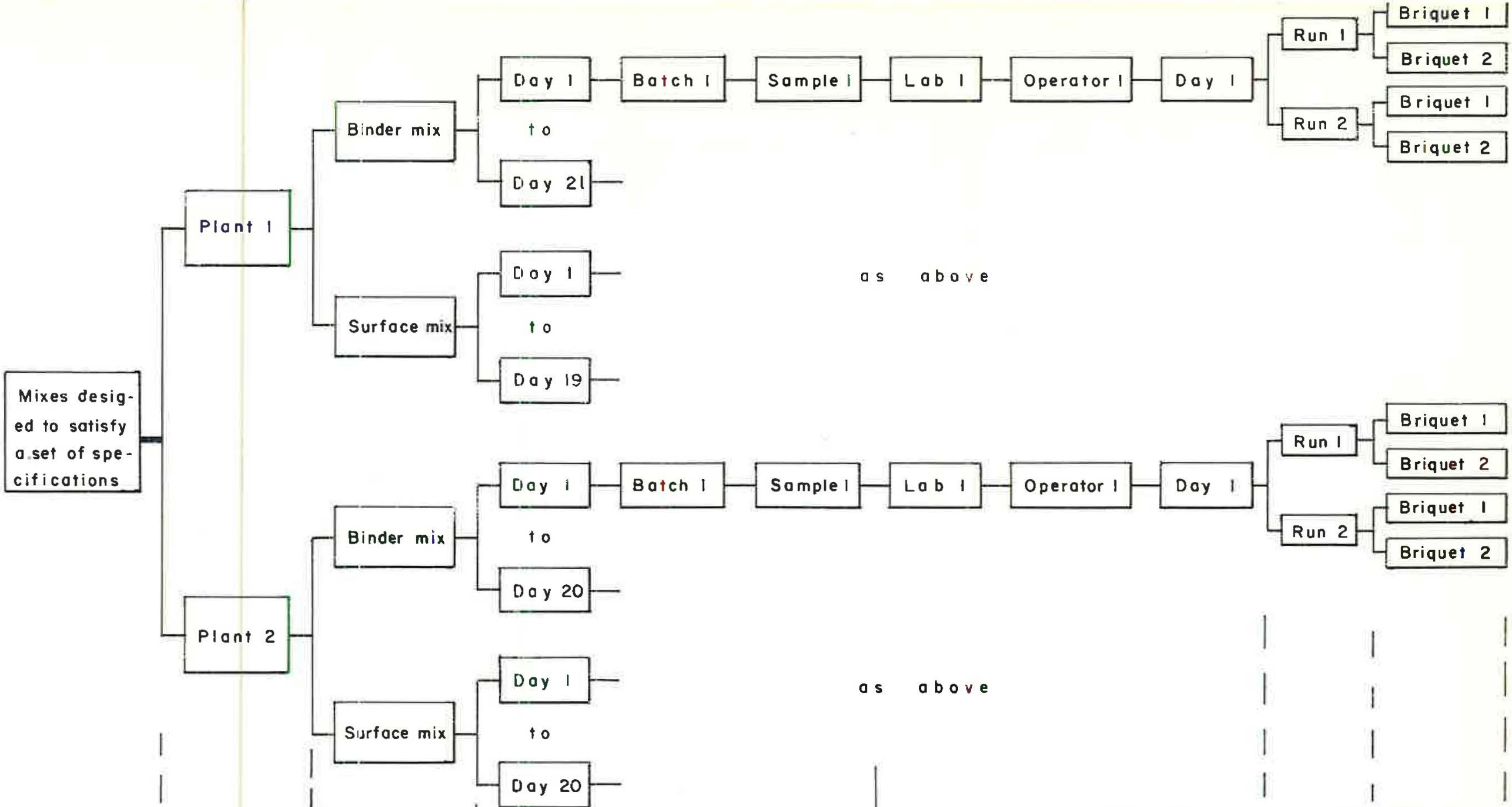


Figure 2. Analysis of variance, laboratory investigation.



Stability	Variability is a fonction of character of materials and ability of plant to produce a uniform mixture	Variability of mix is greatly influenced by mix type.	Significant variation (greatest for surface mix) Day (process), batch, sample and day (testing) components compounded	Not varied	Not significant	Significant variation (greatest for surface mix)

Figure 3. Analysis of variance, field investigation.

TABLE 2

MARSHALL TESTING VARIABILITY OF SIMILAR BITUMINOUS MIXTURE FROM TWO CONTRACTORS[1]

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	Components of Variance[2] Estimated by Mean Square
(a) Analysis of Variance, Contractor C				
Between days	16	2,858,702	178,669	$2\sigma^2_{BD} + \sigma^2_{WR}$
Within day (within a run)	17	692,825	40,754	σ^2_{WR}
	$\hat{\sigma}^2_{WR}$ = 40,800 lb		$\hat{\sigma}^2_{BD}$ = 69,000 lb	
(b) Analysis of Variance, Contractor D				
Between days	17	1,274,481	74,969	$2\sigma^2_{BD} + \sigma^2_{WR}$
Within day (within a run)	18	207,338	11,518	σ^2_{WR}
	$\hat{\sigma}^2_{WR}$ = 11,500 lb		$\hat{\sigma}^2_{BD}$ = 31,700 lb	

[1] Original data taken from routine production test results; two Marshall briquets, one sieve analysis, and one extraction test made on each sample.
[2] BD = between days; WR = within run; F test: $F_{(17, 18)} = \frac{40,754}{11,518} = 3.5$. This is statistically significant at 0.01 level.

approximately the same design specifications. Contractor C's "testing" variability, $\hat{\sigma}^2_{WR}$, is significantly higher than that of contractor D at the 0.01 level of probability.

Thus, it must be recognized that any particular study of the precision of the Marshall test can rarely be applied directly to other laboratories or to material from other suppliers. In addition, the generality of the components of variance reported in the literature is restricted by the mixing methods used to prepare the material for the briquets. For example, in one study (11) each briquet was made from a small batch mixed separately in the laboratory and tested hot, whereas in the investigation reported here four briquets were produced from a 20-lb sample of the contractor's material.

To evaluate some sources of variability of the Marshall and the Rice's vacuum saturation tests, two statistically designed experiments were carried out.

Experimental Conditions

Experimentation was done on two types of mixtures: a 1-in. nominal size binder mix and a ½-in. nominal size surface mix. The experiments were designed to evaluate the effects of four factors, the results to be presented, where possible, as components of variance. Other known factors were controlled by the experimental techniques.

The factors studied were as follows:

1. The Marshall compaction hammers. A two-hammer compaction machine was used. The hammers were designated A and B.
2. The run-to-run variation. Two briquets were compacted per run and two runs were made successively each day.
3. The day-to-day variation. Four consecutive days in each week were devoted to making briquets, the testing being done on the day following.
4. The week-to-week variation. The experiment was carried out over four successive weeks.

The following were closely controlled:

1. Gradation, quality and character of the coarse and fine aggregates and the mineral filler.
2. Grade and character of the asphalt cement.
3. Mixture composition.
4. Preparation and testing technique.

Test Specimens and Testing Technique

Materials. —The mixtures were prepared from three sizes of crushed limestone coarse aggregate, a crushed limestone screening, a natural siliceous coarse sand, a mineral filler, and an 85-100 penetration grade asphalt.

Preparation of Batch Mixes and Compaction. —An 8,000-g batch was prepared each day for each of the two types of mixture in order to provide specimens for four briquets, two Rice's vacuum saturation tests, and one sieve analysis. The procedures used for the preparation of batch mixes and compaction are similar to those described in the

TABLE 3

COMPONENTS OF VARIANCE OBTAINED FROM MARSHALL STABILITY, MARSHALL DENSITY AND RICE'S MAXIMUM DENSITY[a]

Determination	Mix Type	Est. Component of Variance, $\hat{\sigma}^2$							
		Within Run		Between Runs[a]		Between Days		Between Weeks	
		Deg. of Freedom	Value	Deg. of Freedom	Value	Deg. of Freedom	Value	Deg. of Freedom	Value
Marshall stability	Binder	31	36,000	15, 31	21,800[b]	12, 15	7,000	3, 12	0
	Surface	31	3,500	15, 31	550	12, 15	5,000[c]	3, 12	0
Marshall density	Binder	31	33×10^{-6}	15, 31	8×10^{-6}	12, 15	17×10^{-6}	3, 12	0
	Surface	31	20×10^{-6}	15, 31	0	12, 15	21×10^{-6}[c]	3, 12	5×10^{-6}
Rice's max. density	Binder	-	-	-	19×10^{-6}	12, 16	54×10^{-6}	3, 12	0
	Surface	-	-	16	9×10^{-6}	12, 16	26×10^{-6}[c]	3, 12	1×10^{-6}

[a]Within a day.
[b]Significant at 0.05 level.
[c]Significant at 0.01 level.

Asphalt Institute's "Mix Design Methods for Hot Mix Asphalt Paving," except that (a) the mechanical compactor was used and 60 blows were applied on both top and bottom of the specimen (equivalent to 75 blows of the hand-operated Marshall hammers), and (b) two briquets were molded at the same time (or on one run).

Testing Methods. —The testing methods used were (a) for the Marshall Test, ASTM D1559 60T Tentative Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Aparatus (the four briquets were processed together and the tests were made in order of compaction); and (b) for the maximum density test, Rice's vacuum saturation procedure (13).

Test Results and Conclusions

A sample set of the original data from the experiment and specimen analysis of variance calculations are given in the Appendix. The components of variance are summarized in Table 3, from which the following conclusions can be made for the Marshall stability test:

1. Individual test results appeared to approximate a normal distribution as regards "within-run" variability. This was demonstrated by the following statistical test: If the individual results are normally distributed, the within-run differences (range with sign) will also be normally distributed, with mean 0 and standard deviation $\sigma_d = \sqrt{2}\, \sigma_{WR}$, assuming there is no bias between the first and second briquet within each run. These differences were plotted on a control chart and found to fall between the prescribed limits. The cumulative frequencies approximated a straight line when plotted on normal probability paper.
2. There was no statistically significant difference due to hammer positions for either the binder or the surface mix.
3. There was a significant difference, or bias, between the first and second pair of briquets, the first pair being lower by an average of 200 lb for the binder mix and 80 lb for the surface mix. Further investigation failed to locate any cause for this bias. However, as it did not appear in other test series with the same equipment and operator it was concluded that this was nonrepresentative and that no routine correction factor was necessary to adjust for this effect. The operator was told to check routine data for similar signs, but none were observed. In the discussions which follow, this bias is ignored.
4. If "repeatability" is defined as a statistical measure (1σ) of the variation possible between single determinations on a single sample performed by a single operator using the same equipment and technique on different runs on a single day, the repeatability under the conditions prevailing at the City of Montreal Control and Research Laboratory with the specific mixes studied is

Mix Type	$\sqrt{\hat{\sigma}^2_{BR} + \hat{\sigma}^2_{WR}}$
Binder	240 lb
Surface mix	60 lb

Similarly, conclusions regarding the Marshall density test can be drawn as follows:

1. There was no statistically significant difference at 0.05 level due to hammer positions for either the binder mix or the surface mix.
2. There was no bias between the first and second pair of briquets.
3. If repeatability is defined as before, the repeatability under the same conditions is

Mix Type	$\sqrt{\hat{\sigma}^2_{BR} + \hat{\sigma}^2_{WR}}$
Binder	0.0064
Surface mix	0.0045

Similar conclusions regarding Rice's maximum density test can be drawn, as follows: If repeatability is defined as before, the repeatability under the conditions prevailing at the City of Montreal Control and Research Laboratory with the specific mixes studied is

Mix Type	$\hat{\sigma} = \sqrt{\hat{\sigma}^2_{WD}}$
Binder	0.0044
Surface mix	0.0030

Components of Variance

The total test variance for a single result taken within a single laboratory and using a single operator is composed of day-to-day, run-to-run, and within-run variation (assuming the week-to-week component is zero). This is expressed by

$$\sigma^2_{\text{Total test}} = \sigma^2_{BD} + \sigma^2_{BR} + \sigma^2_{WR} \tag{1}$$

The within-run component, σ^2_{WR}, gives an indication of the precision when testing conditions are as uniform as possible, inasmuch as the two briquets in each run are compacted together and follow one another through the rest of the procedure. In addition, this term includes the mixing and sampling variation that can arise between the two briquets.

The between-runs component, σ^2_{BR}, gives an indication of the additional variable element introduced into the procedure as a result of the different conditions prevailing (either on the compactor or later in the procedure) from the first pair of briquets to the second. In addition, any sampling and mixing differences which prevail between the first and second pair of briquets are included in this term.

The between-days component, σ^2_{BD}, gives an indication of the additional variation introduced when the testing is prolonged over several days. Because in the present study a new batch was prepared in the laboratory each day, this between-days component is equivalent to the sum of the between-days-test component, σ^2_{BD}-test, and the between-batch component, σ^2_{BB}. It may reflect mainly the between-batch variation.

The foregoing components give a useful statistical measure of the capability of the testing procedure as currently in use at the City of Montreal Control and Research Laboratory. Examples are given in a later section ("Controlling the Test Procedure and the Supplier's Process") to demonstrate some of the possible applications.

FIELD INVESTIGATION

In the control of bituminous mixtures (or in setting control limits for a given plant) the capacity of the plant to produce a uniform mixture and the unavoidable variation in the composition and character of the mixes must also be evaluated in order to set realistic control limits.

The purposes of this part are (a) to evaluate the ability of two well-controlled plants to produce uniform surface and binder mixes, using different types of aggregates, and (b) to discuss briefly the factors affecting the variability of production.

The major sources of variation in the production and control of asphaltic mixtures are (a) plant equipment, (b) materials, and (c) sampling and testing procedures.

The capability of a plant to produce a uniform mix within the range of the specification is a function of the efficiency of equipment, such as the aggregate feeding system, the heat and draft efficiency of the dryer, the screen efficiency, the temperature control system for aggregate and asphalt, the precision of the scales, and the efficiency of the pugmill. In addition, some variation in bituminous mixtures production may be attributed to human error. However, the batch plant manurfacturer has largely overcome the human error in proportioning by making automatic controls for weighing and proportioning batches.

The materials fed to the asphalt plant constitute another source of variation, the variables being the quality and character of the aggregate and filler, the grade and kind of asphalt cement, and the gradation of the aggregates. In normal production, the quality and character of both asphalt cement and aggregates are usually quite consistent, whereas the grading of the aggregates is subjected to some unavoidable variations.

The third major source of error is inherent in sampling and testing procedures. As in any controlling system, the representative state of the sample is related to the sampling method and the precision of the test is a function of the testing technique.

As shown by the variables previously outlined, the production of asphaltic mixtures is subject to variations and a reasonable allowance must be provided.

The Experiment

The scope and limitations of the field investigation are shown in Figure 3. In brief, binder and surface mixes produced by two local well-controlled plants were sampled during 20 consecutive days. For each type of mix, a hot sample was taken from a single batch (selected at random) and brought to the central laboratory. The sample was then split into eight parts in order to provide subsamples for four Marshall stability (compacted two to a run) and density determinations, two Rice's maximum density tests, and two extraction and grading tests.

The sampling and testing methods and the characteristics of the plants are described in the following.

Sampling and Testing Methods

Unless otherwise specified, the sampling method used was the one described in ASTM Designation D 979-51 under the title "Sampling Plant Mixed Bituminous Mixtures at Place of Manufacture." The testing methods used are: ASTM Designation D 1559-60T Tentative Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus"; extraction by ASTM Designation 1097-58 Standard Method of Paving Mixtures by Centrifuge, except that the dust correction was determined by centrifuging two 100-ml aliquot portions (3); and maximum density by Rice's vacuum saturation procedures (3).

Characteristics of Plants

The mixtures were produced in two plants. Plant I is a fully automatic 6,000-lb capacity batch plant. The dryer capacity is 180 tons per hour (88-in. diameter, 36-ft length). The aggregates are fed to the dryer from five cold bins by calibrated vibration feeders (Syntron). The hot aggregates are screened into five bins and the bitumen is supplied to the pugmill volumetrically. Plant II is a semi-automatic 5,000-lb batch plant. The dryer capacity is 140 tons per hour (diameter 70 in., length 30 ft). The cold feed is calibrated by gate opening and controlled by Weight-O-Matic system. The hot aggregates are screened into four bins and the bitumen is supplied to the pugmill volumetrically.

Both plants were continuously checked and calibrated by the supplier's experienced bituminous engineer.

Mix Type

Two types of dense-graded bituminous mixtures produced by each plant were subjected to investigation--½-in. nominal size surface mix, and 1-in. nominal size base course mix.

Mixtures produced by plant I were prepared from crushed limestone coarse aggregates, manufactured (crushed stone) sand, and natural medium to fine sand; mixtures

TABLE 4

COMPOSITION OF MIXTURES PRODUCED BY TWO PLANTS

Item	Plant I		Plant II	
	Surface Mix	Base Course	Surface Mix	Base Course
Mix composition (%):				
¾-in. crushed stone	13.0	-	20	-
½-in. crushed stone	13.0	17.0	20	-
¼-in. crushed stone	22.5	-	7	11
Screening	-	-	-	15
Manufactured sand	47	29	-	-
Coarse sand	-	41	43	43
Medium sand	-	-	-	-
Fine sand	-	-	10	24
Mineral filler	4.5	13.0	-	7
Grading (%):				
1 in.	-	100	-	100
¾ in.	-	95.9	-	96.6
½ in.	100	84.7	100	80.7
⅜ in.	99.1	78.1	99.0	61.0
No. 4	83.5	54.5	87.1	50.2
No. 8	68.7	39.5	73.5	44.6
No. 16	58.5	32.1	66.1	40.2
No. 30	45.7	23.9	52.8	28.7
No. 50	39.1	20.2	41.6	18.3
No. 100	20.7	11.2	27.5	9.7
No. 200	8.9	4.9	11.0	2.6
Bit. content (%):	6.7	4.8	6.3	4.93

prepared by plant II were prepared from crushed shaly limestone coarse aggregates and natural coarse to fine sand. The compositions of the mixtures are given in Table 4.

Test Results and Conclusions

A sample set of the original data from the experiments and specimen analysis of variance calculations are given in the Appendix. A summary of the components of variance for Marshall and Rice's maximum density test results are given in Table 5, from which the following conclusions can be drawn regarding Marshall stability:

1. Comparing the variations between results obtained from suppliers I and II, the components of variance in general appeared to be higher for supplier I. Tests on within-run components indicate that σ^2_{WR} for supplier I (surface mix) was significantly higher than for supplier II at the 0.05 level of probability. The difference in variation is possibly caused by (a) the character of the materials and (b) the ability of a plant to produce a uniform mixture.
2. Variability of mix seems greatly influenced by mix type. By comparing the within-run variation of binder and surface mixes produced by suppliers I and II, it was found that surface mixes are significantly less variable than binder mixes at the 0.01 level of significance.
3. The bias between the first and second run was not statistically significant at the 0.05 level for any one of the four sets of data.
4. If the repeatability is defined as previously, and the process variability as the square root of the sum of between-days, between-runs, and within-run variances, the following standard deviation values are obtained from the present conditions:

Supplier	Mix Type	Repeatability of Marshall Test (lb), $\sigma = \sqrt{\hat{\sigma}^2_{BR} + \hat{\sigma}^2_{WR}}$	Process Variability (lb), $\sigma = \sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{BR} + \hat{\sigma}^2_{WR}}$
I	Binder	235	305
	Surface	90	185
II	Binder	225	270
	Surface	60	155

TABLE 5

COMPONENTS OF VARIANCE OBTAINED FROM FIELD INVESTIGATION

Determination	Mix Type	Supplier	Estimated Component of Variance, $\hat{\sigma}^2$: Between Days, Deg. of Freedom	Between Days, Value	Between Runs, Deg. of Freedom	Between Runs, Value	Within Run, Deg. of Freedom	Within Run, Value
Marshall stability	Binder	I	20, 20	39,450[a]	-	0	42	54,200
		II	19, 19	22,400[b]	-	0	38	7,900
	Surface	I	18, 18	27,100[a]	-	0	40	49,800
		II	19, 19	18,600[b]	19, 40	2,250	40	3,500
Marshall density	Binder	I	20, 20	45×10^{-6}[a]	-	0	42	29×10^{-6}
		II	18, 18	43×10^{-6}[a]	-	0	40	47×10^{-6}
	Surface	I	19, 19	97×10^{-6}[a]	-	0	38	20×10^{-6}
		II	19, 19	112×10^{-6}[a]	19, 40	8×10^{-6}[a]	40	9×10^{-6}
Rice's max. density	Binder	I	20, 21	250×10^{-6}[a]	21	26×10^{-6}	-	-
		II	19, 20	94×10^{-6}[a]	20	37×10^{-6}	-	-
	Surface	I	18, 19	150×10^{-6}[a]	19	21×10^{-6}	-	-
		II	19, 20	144×10^{-6}[a]	20	11×10^{-6}	-	-

[a]Significant at 0.01 level.
[b]Significant at 0.05 level.

Similarly, for Marshall density:

1. The within-run variation values are similar to those found with the laboratory mixes, there being no statistical difference at the 0.05 level. This constitutes a confirmation of the results given earlier.
2. The estimates for the between-days components of supplier II are more than twice those obtained for supplier I. Considering the fact that the between-days variation in stability of supplier II appears smaller than supplier I, it might be possible (a) that greater variation in stability is not necessarily associated with density variation and (b) that the density variation is associated with the characteristics of the aggregates.
3. The between-days component is large. This might be attributed to the unavoidable grading, bitumen content, aggregate, and plant variations.

Similarly, for Rice's maximum density test:

1. The within-day components are close to the values reported earlier and do not differ statistically at the 0.05 level. This is a confirmation of the repeatability of the test.
2. The between-days variation constitutes the far more important variable. This is associated with the unavoidable day-to-day variations in bitumen content, grading, specific gravity, and process variables.
3. If repeatability is defined as the square root of σ^2_{WD} and the process variability as the square root of the sum of between-days and within-a-day components, the following values are obtained from the field investigation:

Supplier	Mix Type	Repeatability of Rice's Test, $\sigma = \sqrt{\hat{\sigma}^2_{WD}}$	Process Variability, $\sigma = \sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{WD}}$
I	Binder	0.0051	0.017
	Surface	0.0046	0.013
II	Binder	0.0061	0.0115
	Surface	0.0033	0.0125

DISCUSSION OF BITUMEN AND GRADING VARIATION

The present investigation offers an opportunity to discuss some effects of unavoidable bitumen content and grading variations.

Bitumen Content Variation

Previous investigation on bitumen content variation (15) indicates that for a well-controlled production, a total standard deviation, $\sigma_{total} = \sqrt{\sigma^2_{test} + \sigma^2_{process}}$, of 0.2% is normal. Assuming that the test component (σ_{test}) is in the order of 0.1 (1), the process component ($\sigma_{process}$) is then about 0.17. This implies that a natural spread (±2 σ) of ± 0.35% in bitumen content due to process variation is normal for a well-controlled production.

It is well known that a spread of ± 0.35% can considerably affect the physical properties of a given mixture. A typical example of this is shown in the following comparison of two binder mixes of 1-in. nominal size:

Producer	Average Marshall Stability		95% Confidence Interval for True Difference
	B-C 4.4%	B-C 5.2%	
E	2,030	1,870	160 ± 275
F	2,990	2,270	720 ± 195

The stability values given are the averages of five runs taken from five different batches. From this table it can be seen that by increasing the bitumen content from 4.4 to 5.2 (or 0.8%) the average stability values have decreased 160 lb for producer E and 720 lb for producer F. It should be noted that, based on previous data, the difference between results obtained from producers E and F is reliable within ± 380 lb (95% confidence level).

From the foregoing it may be concluded that, depending on the composition of the mix and the type of aggregate used, the unavoidable bitumen content variation can be an important component of the plant process variation.

Grading Variations

One of the most important factors in the production of bituminous mixtures is the variation in the aggregate gradation. It is well known that physical characteristics of bituminous mixtures are influenced by gradation variations.

In general, crushed aggregates are less variable than natural aggregates, the variations in grading of the crushed stone being related to the efficiency of crushing and screening operations, which in turn are related to the characteristics of the aggregate, rate of production, weather conditions, and many other factors.

The fine aggregates used in bituminous mixtures are usually screened or unscreened natural sand, which is more or less variable depending on its origin.

Grading specifications are generally stipulated for the coarse aggregate, the fine aggregate, and the combined aggregate. A typical example of this is the maximum permissible variation limits from job-mix formulas (Table 6), recommended by the Asphalt Institute for all types of mixes.

Test results from the AASHO Road Test and the present study (Table 7) indicate that limits such as those given in Table 6 are too narrow. It is believed that more realistic limits must take into account the relative importance of coarse and fine aggregate fractions.

The influence of grading variation on the physical properties of the mix is different for different types of mixtures. As a general rule, if the grading variation increases the density of the mix the stability will increase, the voids in mix will decrease, and the voids in mineral aggregates will also decrease.

TABLE 6

ASPHALT INSTITUTE PERMISSIBLE VARIATION FROM JOB-MIX FORMULA

Sieve Size	Permissible Variation (% wt. of tot. mix.)
No. 4 +	5.0
No. 8	4.0
No. 30	3.0
No. 200	1.0

Figure 4 shows typical results obtained from a well-controlled plant. It illustrates how the stability values varies with the unavoidable bitumen content and grading variations.

Maximum Size Effects

The statistical analysis of the present laboratory and field investigations offers an opportunity to discuss the maximum size effects.

Table 8 gives the between-days variations for field Marshall stability. It is interesting to note that for both supplier I and II, these components for between-days variation, which reflect the capability of a plant

TABLE 7

GRADING VARIATIONS: AASHO ROAD TEST AND PRESENT FIELD INVESTIGATION

Sieve	Asph. Inst. Permissible Variation (%)	AASHO Road Test				Present Field Investigation			
		Binder Mix		Surface Mix		Binder Mix		Surface Mix	
		Mean	σ	Mean	σ	Mean	σ	Mean	σ
1 in.	-	100	-	-	-	100	-	-	-
¾ in.	-	96	2.21	100	-	96.6	1.83	-	-
½ in.	-	76	3.30	92	2.43	80.7	2.94	100	-
⅜ in.	-	57	2.71	81	3.17	61.0	2.48	99.0	1.32
No. 4	± 5	36	2.18	63	4.04	50.3	2.95	87.5	0.79
No. 8	-	-	-	-	-	44.6	2.73	75.5	1.85
No. 10	± 4	25	1.29	46	2.99	-	-	-	-
No. 16	-	-	-	-	-	40.2	2.44	68.0	1.83
No. 20	-	19	1.03	34	1.68	-	-	-	-
No. 30	± 3	-	-	-	-	28.7	2.07	53.0	2.01
No. 40	-	13	0.98	22	2.06	-	-	-	-
No. 50	-	-	-	-	-	18.3	1.38	41.4	2.13
No. 80	-	8	0.81	13	1.07	-	-	-	-
No. 100	-	-	-	-	-	9.7	1.11	25.7	1.85
No. 200	± 1	4.3	0.49	5.9	1.16	2.6	0.50	10.2	0.83
A. C.	± 0.3	4.2	0.13	5.2	0.18	4.93	0.33	6.35	0.22
Nb of test		127		96		40		40	

to produce a uniform mixture, appear relatively unaffected by the mix type.

Table 9 shows that for both the laboratory and field investigations (and for both suppliers I and II), the spreads of Marshall stability test results of the 1-in. nominal size binder mixes are nearly three times wider then those obtained with ½-in. nominal size surface mixes. Assuming that the only major difference between the surface and binder mixes is the nominal size of the aggregates, the tests results clearly indicate the maximum size effect on the repeatability of the Marshall tests.

TABLE 8

COMPARISON OF MARSHALL STABILITY BETWEEN-DAYS RESULTS

Supplier	Mix Type	$\hat{\sigma}^2$
I	Binder	39,450
	Surface	26,900
II	Binder	22,400
	Surface	18,600

TABLE 9

REPEATABILITY OF MARSHALL TEST

Supplier	Value of $\hat{\sigma}$			
	Surface Mixes (½-in. max.)		Binder Mixes (1-in. max.)	
	Lab. Invest.	Field Invest.	Lab. Invest.	Field Invest.
I	Not tested	90	Not tested	235
II	60	60	240	225

DISCUSSION OF TOTAL VOIDS IN MIX

In the preceding the components of variance were given for Marshall density and Rice's maximum density. Defining voids in the mix by

$$V_m = (1 - G_b/G_r)^{100} \tag{2}$$

in which G_b is the average of two Marshall bulk densities and G_r is the Rice's maximum density (based on one test), the components of variance for voids in the mix of the field investigation are given in Table 10.

In this table it is interesting to note that the between-days component, which is related to the process variation, appears much higher than the within-day component, which reflects the testing precision.

The total standard deviation and normal spread shown in Table 11 clearly indicate that a spread of 3% in "voids in mix" can be expected from a well-controlled production if the testing is carried out as described earlier.

EFFECT OF INHERENT VARIABILITY ON MIX DESIGN AND SPECIFICATION

The repeatability of the Marshall (stability and density) and Rice's maximum density tests have been estimated, and the plant variation that occurs for a well-controlled production process has been analyzed in the foregoing. It is the prupose of this section to study some effects of these variations on (a) the significance of laboratory mix design formulas and (b) the specification limits.

TABLE 10

COMPONENTS OF VARIANCE FOR VOIDS IN MIX, FIELD TEST

Component[1]	Mix Type	Variance: Supplier I: Deg. of Freedom	Supplier I: Value	Supplier II: Deg. of Freedom	Supplier II: Value
$\hat{\sigma}^2_{BD}$	Binder	20, 21	0.2010	19, 20	0.3150
	Surface	18, 19	0.1400	19, 20	0.3200
$\hat{\sigma}^2_{WD}$	Binder	21	0.1050	20	0.0960
	Surface	19	0.0680	20	0.0310

[1] BD = between days (includes between-batch variation);
WD = within days.

TABLE 11

STANDARD DEVIATION AND NORMAL SPREAD FOR VOIDS IN MIX, FIELD TEST

Item	Mix Type	Supplier I	Supplier II
Standard deviation,			
$\hat{\sigma}_{tot} = \sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{WD}}$	Binder	0.55	0.63
	Surface	0.46	0.67
Total spread, $6\hat{\sigma}_{tot}$	Binder	3.3	3.8
	Surface	2.8	4.0

Significance of Laboratory Mix Design Formulas

The first step in mix design is to determine in the laboratory which combinations of aggregates and asphalt would give the required stability and durability. Usually, in determining the optimum asphalt content for a particular blend or gradation of aggregates by the Marshall method, a series of test specimens is prepared for a range of different asphalt contents so that the test data curves show a well-defined optimum value. A single batch is prepared for each ½ percent increment of asphalt content, and triplicate test specimens are usually prepared from each batch. The next step is to go to the plant, calibrate the cold feed system, and find the optimum bin proportions that will produce a combined gradation conforming as closely as possible with the mix design formula. Once the job-mix formula is set, the characteristics of the produced mixtures must be kept within prescribed specification limits to assure uniformity of the mixture.

In an earlier section it was demonstrated that the most important component of variance in bituminous production is the day-to-day variation, especially for surface mix, where the repeatability of Marshall test is good. This day-to-day variation, which reflects the capability of a plant to produce a uniform mix, is usually unknown during the design stage. It follows that the mix design is only a rough estimate of the end results.

Figure 4 shows the scatter obtained when stability values are plotted against bitumen content for samples from a well-controlled production plant. The data shown come from a binder mix produced by supplier II. Sieve analyses of the samples indicate that all grading results fall within $\pm 3\sigma$ limits (the standard deviation being as given in Table 7, Col. "σ, Binder Mix, Present Field Investigation"). Examination of Figure 4 allows the following comments:

1. The individual stability values vary within a wide range. This is associated with unavoidable grading, bitumen content, plant, and testing variations.
2. The mix-design (laboratory) stability values obtained with 4.5 and 5.0 percent asphalt cement give a poor estimate of the production stability-asphalt content relationship. This is because in the mix design data only one laboratory batch is considered, whereas in routine production grading and other factors affecting stability vary from batch to batch.
3. Once estimates of process and testing components of variance have been obtained it may be possible, by using procedures described later herein, to determine the precision of inferences based on the laboratory design results.

Some further work may be necessary, however, to establish the connection between results obtained from laboratory-prepared mixtures and those from production batches. It is quite possible, for example, that the relationships between bitumen and stability indicated by the laboratory design tests may not be valid for other grading distributions lying within acceptable limits (see Fig. 4). In this case all that could be done with the laboratory results would be to estimate the range of production properties which should result for any mix design formula, given a specific supplier, a prescribed amount of routine testing, and a set of bitumen and grading limits.

Specification Limits

In setting realistic specification limits it is essential (a) to specify in detail the method of testing and the calculation procedures; (b) to ascertain limits for the properties, within which the material may be considered acceptable; (c) to consider the natural spread of the process under maximum control, taking into account the volume of sampling and testing to be performed; and (d) to give specific decision rules whereby material can be accepted or rejected, making provision for allowable sampling and testing fluctuations. The statistical analysis of test results from the AASHO Road Test and the present investigation offers an opportunity to underline the importance of items (a) and (b).

From Table 12, a summary of specification limits, field test results and assumed components of variance for both the AASHO Road Test and supplier II of the reported field investigation, it can be seen that the natural limits of the Marshall stability and total-voids-in-mix values cannot be directly compared.

If it is assumed that the standard deviation values for voids for both the present investigation and the AASHO Road Test are based on same components, $\sqrt{\sigma^2_{BD} + \sigma^2_{WD}}$, they cannot be directly compared, because the method of calculation differs. For the

- ▲ Routine production binder mix with a target job-mix bitumen content of 4.6%
- • Routine production binder mix, same grading but a target bitumen content of 5.0%
- o Original laboratory averages obtained for mix design
- ● Average point for production results

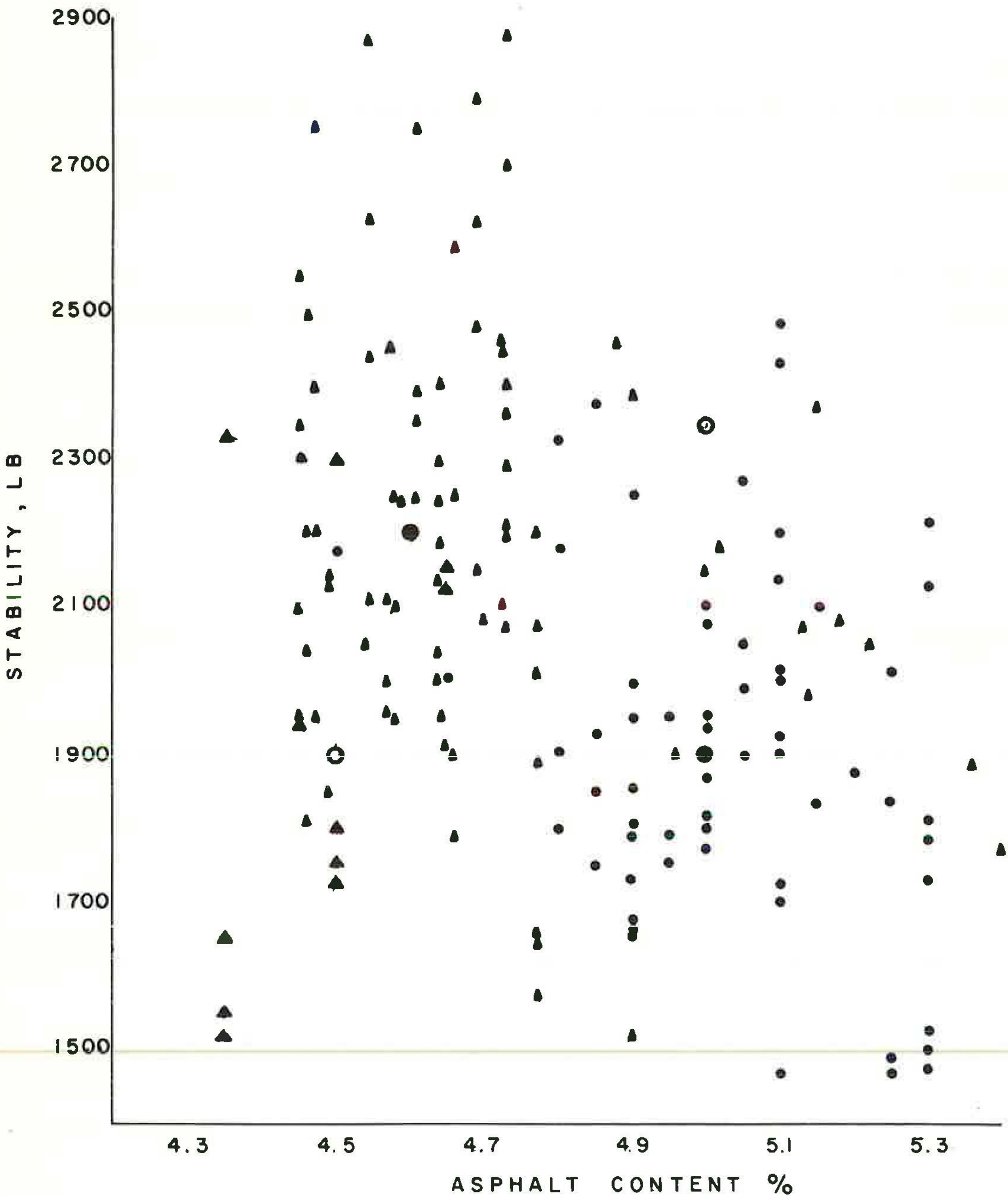


Figure 4. Comparison of routine laboratory design results with stability and asphalt content measurements from the controlled process.

TABLE 12
SUMMARY OF MARSHALL TEST RESULTS, PRESENT FIELD INVESTIGATION AND AASHO ROAD TEST

Investigation	Mix Type	Stability (lb) Mix Design Requir.	Mean Value	Std. Dev.	Components	Voids (% tot. vol.) Mix Design Requir.	Mean Value	Std. Dev.	Components
Present	Binder[1]	1,500[2]	1,919	305	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{BR} + \hat{\sigma}^2_{WR}}$	2 - 5	2.99	0.63	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{WD}}$
	Surface[1]	1,500[2]	1,796	185	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{BR} + \hat{\sigma}^2_{WR}}$	2 - 5	3.3	0.67	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{WD}}$
AASHO Road Test	Binder	1,500 - 2,500	1,770	190	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{BR} + \left(\hat{\sigma}^2_{WR}\right)/2}$	4 - 6	4.8	0.52	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{WD}}$
	Surface	1,500 - 2,500	2,000	125	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{BR} + \left(\hat{\sigma}^2_{WR}\right)/2}$	3 - 5	3.6	0.43	$\sqrt{\hat{\sigma}^2_{BD} + \hat{\sigma}^2_{WD}}$

[1]Supplier II. [2]Minimum.

present investigation, the voids in mix is defined by Eq. 2, in which G_r is the Rice's maximum density (based on one test). For the AASHO Road Test, V_m is defined by a similar equation in which G_r is replaced by G_a, the specific gravity of the mix calculated by using the "apparent" specific gravity of aggregates and the bulk specific gravity of the bitumen. It must be noted that if it is assumed that G_a is constant (as is frequently done), the standard deviation value will only reflect the Marshall bulk density variation. This shows that in setting specification limits it is essential to describe the test procedure and calculation method to be used.

The observed natural limits are related to the volume of sampling and testing. It is thus essential to decide if the limits are for single test results or for means of n results based on a specified number of runs. Examination of the standard deviation values given in Table 12, and their related components of variance, shows this.

For the present field investigation, the specification requires that a minimum stability value of 1,500 lb must be met by any single test value (one briquet per batch).

This implied σ (for natural spread) $= \sqrt{\sigma^2_{BD} + \sigma^2_{BR} + \sigma^2_{WR}}$.

For the AASHO Road Test the specification requires that the average stability value of two briquets prepared from any truck sample must fall between 1,500 and 2,500 lb. Assuming that the sample is taken from a batch each day, σ (for natural spread) =

$$\sqrt{\sigma^2_{BD} + \sigma^2_{BR} + \left(\sigma^2_{WR}\right)/2}.$$

Figures 5 and 6 compare the specification limits and the natural spread of the AASHO Road Test data and the present field investigation test results. Examination of Figure 5 gives rise to the following comments:

1. In the cases of both surface and binder mixes of supplier II, as well as the binder mix of the AASHO Road Test, the specification limits do not, even though the two processes are in a "state of control," coincide with the natural limits.
2. In the case of supplier II, if the minimum specification requirement of 1,500 lb is to be observed, the process mean should be maintained 3σ above the minimum specification limit (that is, $\bar{x}$ = 2,300 lb for binder mix and $\bar{x}$ = 2,000 lb for surface mix, $\bar{x}$ being the process mean).
3. In the case of the AASHO test binder, where upper and lower specification limits are set, it can be seen that the process even when "in control" cannot supply 100 percent acceptable product in its present state. This is because the natural spread of the "in control" process (6σ = 1,140 lb) is larger than the range of the specification limits (2,500 - 1,500 = 1,000 lb). In such a case if it is desired to minimize off-specification material, the process mean should be kept at the specification mean (2,000 lb).
4. In the case of the AASHO surface mix, the natural spread lies within the specification limits. If a reduction in stability value reduces the cost of the mixture, and if the specification is to be observed, it is thus advantageous to keep the process mean 3σ (375 lb) above the lower specification limit.

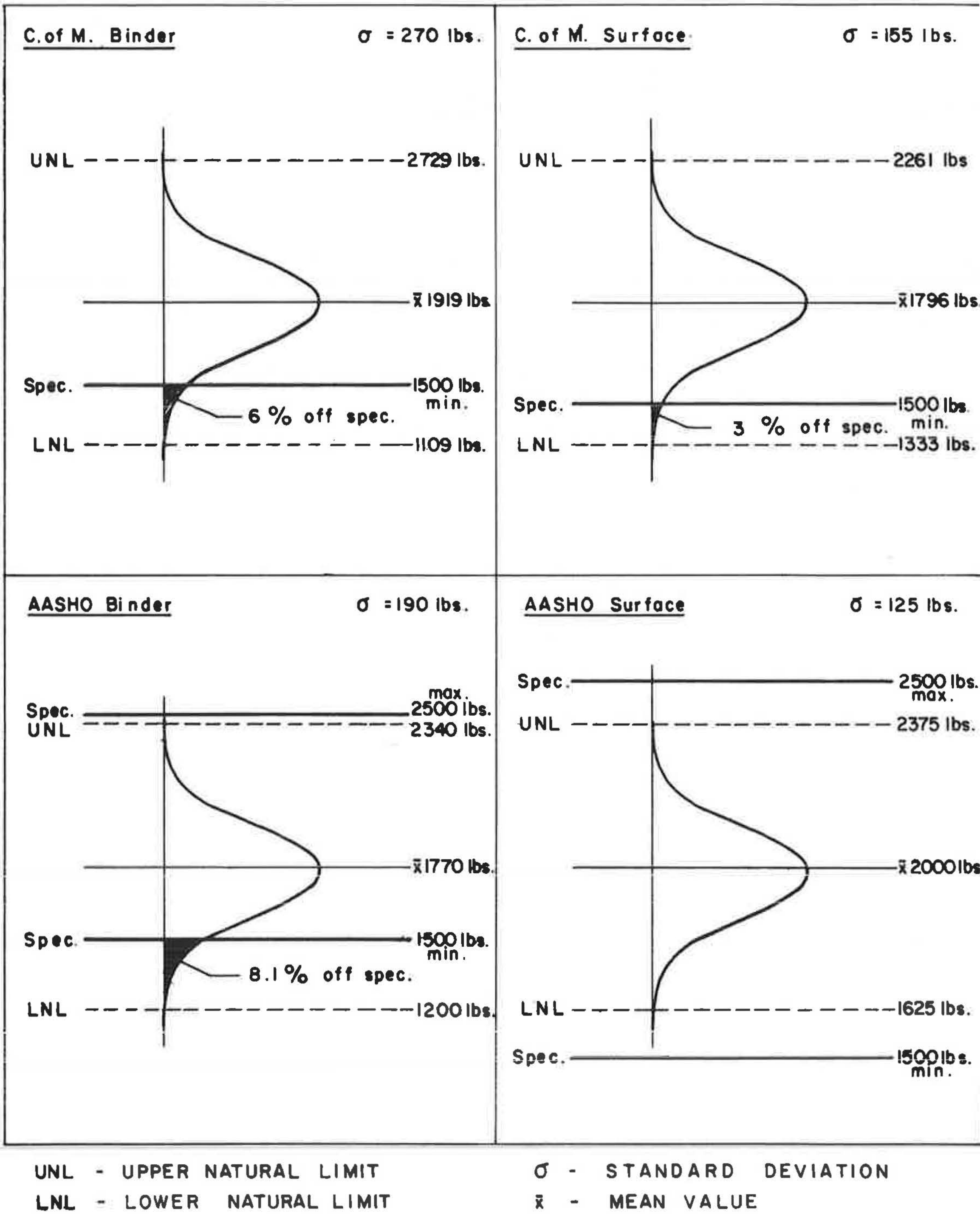


Figure 5. Process variation vs specification limits, marshall stability.

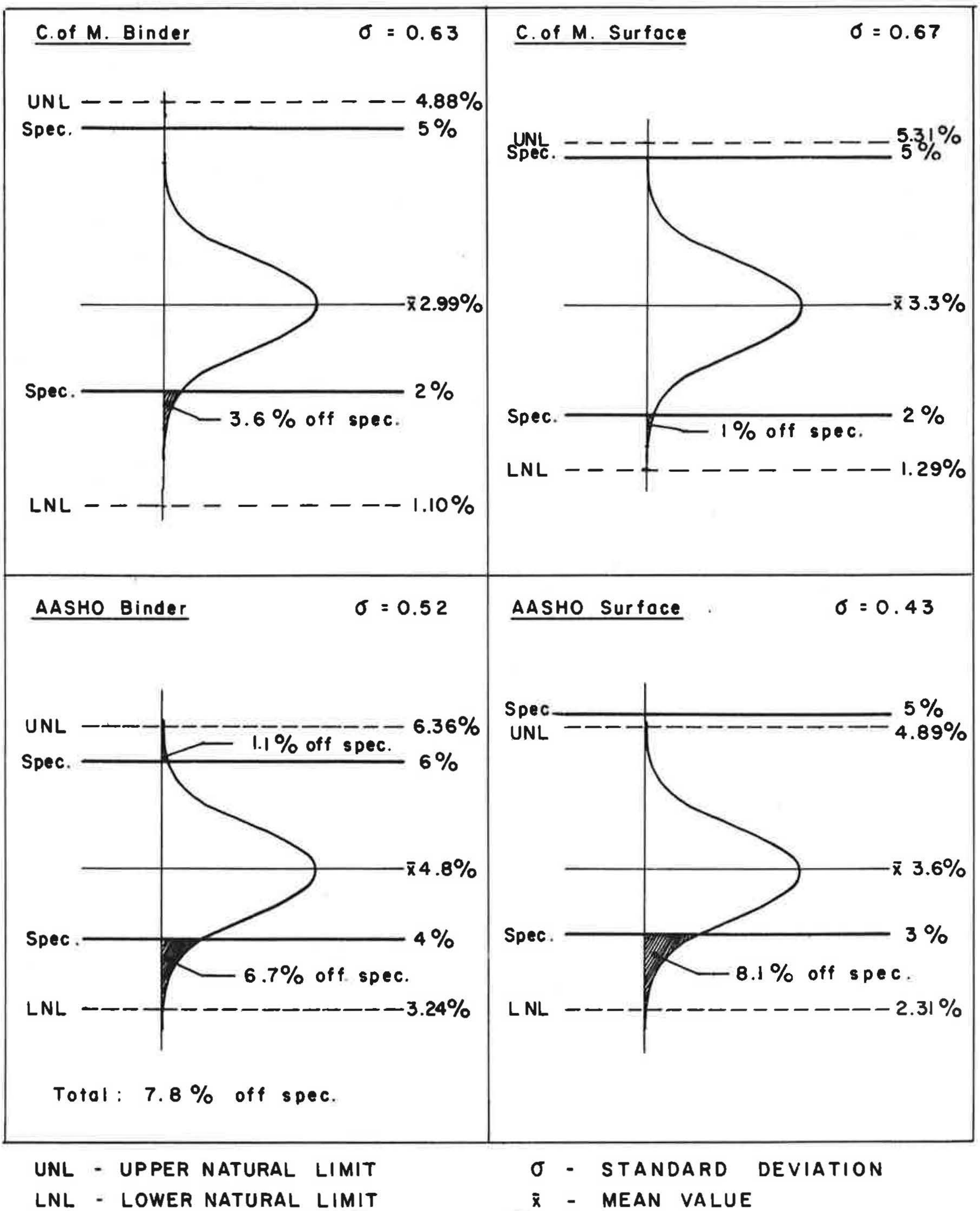


Figure 6. Process variation vs specification limits, total voids in mix.

Figure 6 gives rise to the following comments:

1. Both City of Montreal and AASHO's limits (C of M, 2 to 5%; AASHO, 4 to 6% for binder and 3 to 5% for surface) lie inside the natural spread. This means that for the sampling, testing and calculation conditions described earlier the specification limits are too tight or some tolerance must be allowed to take care of the unavoidable off-specification portions.

2. In both cases the standard deviation values are close to 0.5 percent; in other words, in normal production a natural spread of at least 3 percent in voids is unavoidable. This implies that if it is desired to keep the production mixture within the specification limits, the minimum range of specification limits should be at least 3 percent and the process mean should fall in the center of this range.

CONTROLLING TEST PROCEDURE AND SUPPLIER'S PROCESS

Although the Marshall test (stability) is used for illustrative purpose, the following discussion could be applied in principle to other tests. Where a number of interrelated tests are used to decide the acceptability of a product, "multi-variate" control charts (4) may be used; however, only single-variate charts are described here. The discussion is divided into subsections dealing with (a) control within the laboratory, including establishing the capability of the testing procedure and maintaining control of the testing procedure; and (b) control over the supplier's material, including establishing the capability of the supplier's process and maintaining control over the supplier's process.

Control Within Laboratory

Establishing Capability of Testing Procedure. —Before meaningful sampling and testing frequencies can be established, it is necessary to establish the capability of the testing procedure and the "within-laboratory" factors which may influence the results. This capability is reflected by the "accuracy" of the test method, which may be defined as the extent to which test results may differ from the "true" or standard reference value. It should be noted that this definition includes the laboratory bias (should one exist), as well as the variability inherent in the procedure (21, 22).

The laboratory "bias" is that consistent difference separating the laboratory average either from the "true" value or from an acceptable reference value. This latter value is sometimes established by equating it to the grand average in a round robin program in which several laboratories carry out tests on similar material.

The variability of the testing procedure is best expressed in terms of components of variance determined by studies similar to that described earlier in this paper. Components of variance, as the name implies, are measures of the variability which may be ascribed to various sources, such as different operators, different pieces of test equipment, and even, in some cases, different stages of the test procedure itself. A more complete discussion of this subject is given elsewhere (29 - 35). Because of the nature of the calculations required, it is possible to build up information regarding these components through routine testing and small limited studies, as well as through more comprehensive programs such as those described here. Unfortunately, estimates of these components can be subject to relatively large variability themselves and those based on a few results are not reliable. The reliability may be increased, however, by combining several estimates of the same components. Where there are only a limited number of operators and testing machines in the laboratory, consistent differences in the average level of results, which may be ascribed to these factors, are often considered as biases, which are added or subtracted, as an adjustment, whenever the use of the data requires it.

Preliminary studies of test capability often disclose magnitudes of bias or variability which are unsatisfactory and which make it necessary to tighten the standard practices or refine the procedure.

Because an earlier section of this paper was concerned with the sources and magnitude of the variability of the test procedure and did not include a reference point for the determination of bias, the latter is not discussed more fully here, although its importance must not be discounted.

Use of Components of Variance.—Before the components of variance can be used to make probability statements about the precision of the test, it is necessary to ascertain that the individual results from specimens (briquets) taken from a sample tend to cluster about some single central value. (If this is not so, the test in its present state is meaningless.) Statistical theory can then be used to calculate limits about single test results, or averages of test results, such that the long-run average of the sample (or batch, if sampling components are included) will be bracketed by these limits at a given level of statistical confidence. For this estimate it is assumed that the level of control is maintained and that extraneous sources of variation are not introduced.

Several examples can now be given to demonstrate the foregoing.

Example 1: It was found earlier that for Marshall stability (binder material) $\hat{\sigma}^2_{BR}$ = 21,800 lb, σ^2_{WR} = 36,300 lb, and the individual results approximated a normal distribution.

(a) Therefore, within a particular day it can be said with 95 percent confidence that a single test result will be within $\pm 2\sqrt{\sigma^2_{BR} + \sigma^2_{WR}} = \pm 480$ lb of the long-run average of the sample on that day.

(b) If n briquets (n-even) are made, tested on one day (two to a run), and the results averaged, it can be said with 95 percent confidence that the average will be within $\pm 2\sqrt{\dfrac{2\sigma^2_{BR} + \sigma^2_{WR}}{n}}$ lb of the long-run average of the sample on that day.

(c) Again, suppose it is necessary to compare two samples on the same day for stability. If n briquets are made and tested (two to a run) for each sample, the averages for the two samples would have to differ by at least $\pm 2\sqrt{2}\sqrt{\dfrac{2\sigma^2_{BR} + \sigma^2_{WR}}{n}}$ before a difference statistically significant at the 0.05 level could be claimed. (This difference would, however, not necessarily imply a difference between the parent mixes, as explained later.)

From this it can be seen that the limits of uncertainty about any quoted result or average can be reduced as required by increasing the number of briquets tested.

If there is assurance that the between-days testing component can be neglected, the limitation of single days in the preceding discussion can be removed. At present the data are insufficient to make a decision on this question.

Other statistical formulas are available which will enable the experimenter to calculate, in advance, the number of briquets to test and average for each sample, to meet pre-calculated risks for each of the two errors possible:

Error 1—No statistically significant difference is found, whereas in reality a difference of importance exists.

Error 2—A statistically significant difference is found, whereas in reality no difference exists.

Alternatively, a sequential sampling plan could be devised which would be more economical for the problem just described. The briquets would be made and tested in pairs (one from sample A and one from sample B). As the results were obtained a function of the accumulated differences would be plotted on a graph. At each stage the graph would indicate one of the following three courses of action:

1. Decide that a difference does exist.
2. Decide that a difference does not exist.
3. Test another pair of briquets.

Finally, a decision would be made which would be subject to the predetermined risks.

It should be noted that a difference between sample A and sample B does not necessarily imply a difference between batch A and batch B, because there may be (a) a between-sample component of variance and/or (b) a between-batch component of variance (which is introduced when a number of batches are made according to the same formula). Either of these additional components could be sufficient to account for the difference found. That there is such a between-batch component was suggested by the highly significant between-days component for the surface mixture in the study described in the first section of this paper, inasmuch as this variation could conceivably be ascribed to the fact that a new batch was prepared each day. If comparisons between laboratory batches are carried out frequently it would be desirable to estimate more precisely the value of this between-batch component by preparing several batches a day, testing and repeating for a number of days according to a statistical design.

Once the magnitudes of the between-samples and between-batch components have been established, this information can be used to extend the argument of Example 1. For instance, suppose that the laboratory mixes were small enough that the term "sample" had no real meaning (compared with the sample obtained in the field from a production batch or truck load) and that the between-batch component, σ^2_{BB}, was 15,000 lb. The situation outlined in Example 1 (c) may now be generalized. Two binder formulas, A and B, are to be compared for stability, one batch being prepared for each. If n briquets are made and tested for each batch on the same day, the difference between the observed averages would have to exceed

$$\pm 2\sqrt{2}\sqrt{\sigma^2_{BB} + \frac{2\sigma^2_{BR} + \sigma^2_{WR}}{n}} \quad \text{or}$$

or

$$\pm 2\sqrt{2}\sqrt{15,000 + \frac{43,600 + 36,300}{n}}$$

before a difference statistically significant at the 0.05 level of probability could be claimed.

Maintaining Control of Testing Procedure.--Because of the tendency for equipment to wear, and of operators to relax their observance of standard instructions, it is necessary to carry out routine checking procedures.

Where control samples of known value are available, these may be introduced with predetermined frequency under the guise of a routine production sample. Control charts and other statistical procedures can be used to signal significant deviations from standard conditions.

Where such control samples are not available, shifts of the laboratory mean (due to a deterioration of the hammers or other causes) are more difficult to detect and only become apparent when a cross-check is made with another laboratory. It is possible, however, to maintain a weak measure of control on the variability of the testing procedure. This might be done as follows:

Where it is the practice to make two briquets for each production sample, compacting one on hammer A and the other on hammer B, the briquets would be numbered 1 and 2, 1 always being assigned to the briquet compacted on hammer A. If there is no difference between the hammers, the difference between the results (No. 1 - No. 2) should be distributed normally, with average 0 and variance $2\sigma^2_{WR}$ (assuming the individual test results are normally distributed). A regular Shewhart control chart (36) can then be prepared with center line 0, upper control limit $+ 3\sqrt{2}\,\sigma_{WR}$ and lower control limit $- 3\sqrt{2}\,\sigma_{WR}$. The ability of this chart to signal an increase of given magnitude in the variation or the bias can be calculated. It will be noted, however, that this method depends on the differences between duplicates, which are notoriously unreliable in providing a realistic measure of the overall testing variability.

Example 2: Suppose σ_{WR} = 190 lb (from binder study, first section). Figure 7 shows how the 32 differences plotted chronologically fall between the limits + 800 and - 800 lb. It can be calculated that the following conditions will generate an alarm signal (a single point outside the limits) after the specified length of time:

Condition	Average Number of Points Plotted Before One Falls Outside Limits
1. Normal	300
2. σ_{WR} increase by 25%	60
3. σ_{WR} increase by 50%	20
4. Hammers differ by a bias of 200 lb	80

The time interval can be decreased by adding rules for "runs" but this will also increase the number of erroneous signals arising from condition (1).

Control Over Supplier's Material

Establishing Capability of Supplier's Process. —As in the case of the test procedure itself, preliminary studies must be carried out on the supplier's process before the most efficient sampling patterns and frequencies can be determined. In particular, some attempt must be made to determine the variability within a processed batch, between batches within a day, and between days. This is not as difficult as it may appear and there is a good possibility that once these components of variance have been established and the process directed into a state of statistical control, not only will knowledge be increased with respect to possible trouble areas, but efficient sampling plans also can be set up which will be applicable for similar situations elsewhere.

Some laboratories make a practice of combining samples from several different batches to form a composite sample from which one or two briquets are prepared. The implications of this procedure should be studied thoroughly, because there must be some doubt about the combined effect of several mixes taking the size of the briquet into account.

In this preliminary study it may be found that the variability of the process, although predictable, is unsatisfactorily large for the user's purposes. In such cases the supplier will have to find measures to reduce it before his material can be considered acceptable.

After the components of variance have been established and the tolerances calculated within which the supplier's average will be permitted to vary, the risks for the control chart can be set and the sampling and testing frequency determined. The limits for the control chart are based on this preliminary work and so are determined before the routine testing commences.

In the absence of an analysis of variance study, statistical control can be established on a more empirical basis by the method used (9) where the control limits are set after a number of routine test results have been obtained. This method is valid, but in most applications it does not (a) provide the insight into the supplier's process which results from the more intensive preliminary study, or (b) permit optimum sampling and testing frequencies to be devised.

Maintaining Control Over Supplier's Process. —In normal manufacturing practice it is the supplier's responsibility to maintain the control charts, producing them on request for the customer's inspection. Where this is impractical and the customer is obliged to carry out some sampling and testing for his own protection, this should be specifically designed to provide, at minimum cost, the protection desired, with satisfactory risks from both the customer's and the supplier's viewpoints.

The crux of the argument for statistical control is that protection can be achieved most economically by (a) establishing that the process is predictable and (b) providing measures (the control chart) to indicate when this assumption is no longer valid or when the process has shifted into an unsatisfactory region. The alternative is to have no objective assessment of the nature of the process variations and either (a) proceed on an intuitive basis of acceptance and rejection (which is arbitrary and unsatisfactory on scientific grounds) or (b) try to maintain protection by large-scale sampling and testing.

Shewhart Control Chart. —Where the process has shown itself to be relatively stable and in a state of statistical control, the Shewhart control chart may be used to provide

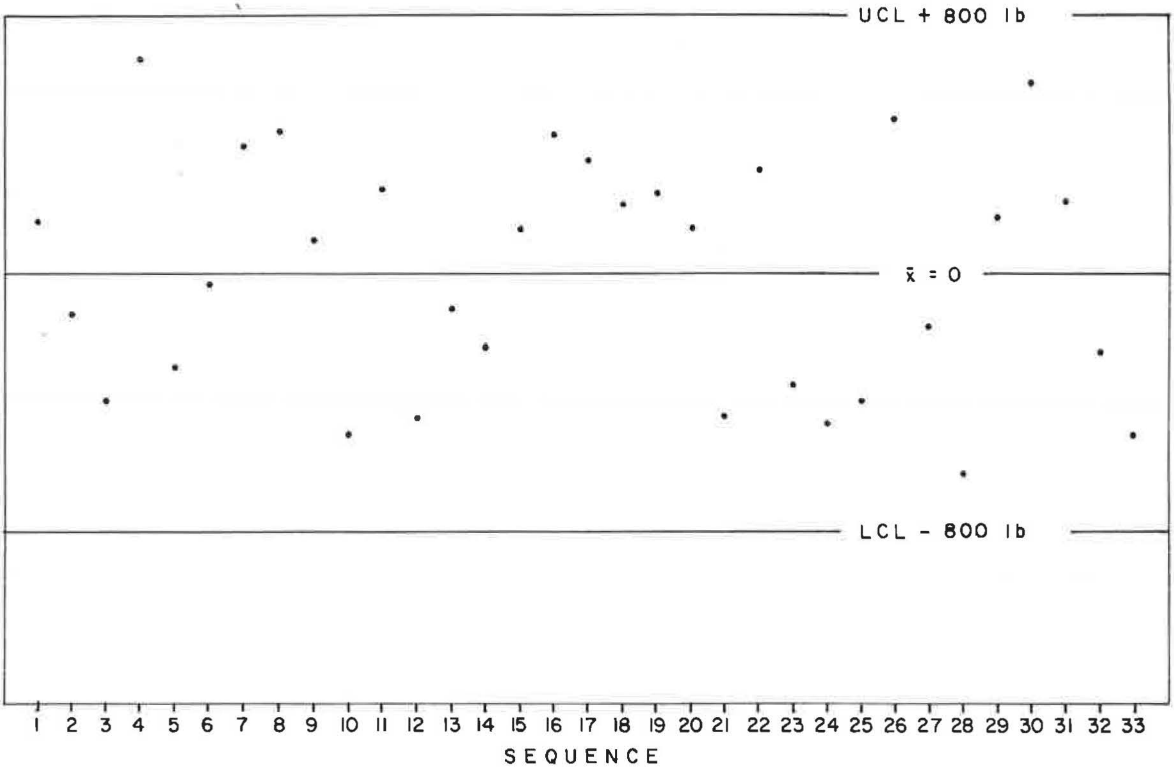


Figure 7. Construction of a control chart for within-laboratory variation.

assurance that the mean of the supplier's process has not shifted (as a result of a different source for the aggregate, say) to an unsatisfactory level, or that the variability of the material has not changed. Little assurance can be provided under this system that unsatisfactory batches, or even unsatisfactory groups of batches occurring sporadically, will be detected.

Before the sampling plan can be set down, two points must be considered: (a) How soon after the shift is it necessary to have a signal that the supplier's material is unsatisfactory? and (b) What protection is the supplier to have against the situation where he is told to adjust a process (or accept a penalty) when the process in reality is providing acceptable material?

Once these have been agreed to, the sampling frequency can be calculated and the chart limits set. The reasoning and mathematical procedures involved are best illustrated by an example.

Example 3: Suppose the major consideration for Marshall stability for a particular binder material is that no batch average falls below 1,500 lb. Therefore, any batch with a true average Marshall stability value less than 1,500 lb will be called unacceptable.

The suppliers process is "in control" and running at an average level of 1,900 lb. It has been established that $\sigma^2_{\text{between batches}} = 20,000$ lb, $\sigma^2_{\text{between samples}} =$ 20,000 lb, $\sigma^2_{\text{between runs}} = 21,800$ lb, and $\sigma^2_{\text{within run}} = 36,300$ lb (with negligible between-days testing variation).

Consider the distribution of batch averages under this system. Assuming a normal pattern this is depicted in Figure 8, from which it is evident that the process average is on the borderline for acceptability because any downward shift would generate unacceptable batches.

The control chart for daily averages would be constructed with center line 1,900 and limits ± 3

$$\sqrt{\frac{\sigma^2_{BB}}{n_1} + \frac{\sigma^2_{BS}}{n_1 n_2} + \frac{2\sigma^2_{BR} + \sigma^2_{WR}}{n_1 n_2 n_3}}$$

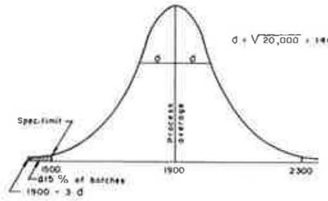


Figure 8.

assuming the product $n_1 n_2 n_3$ is even, n_1 being the number of batches sampled each day, n_2 the number of samples taken from each batch, and n_3 the number of briquets made and tested from each sample.

It is assumed that samples are kept separate (that is, there are no multi-batch composite samples).

The selection of values for $n_1 n_2$ and n_3 will depend on the following factors:

1. The cost of each sampling stage.
2. The cost of testing.
3. The level of protection required.
4. Other factors of practicability.

Suppose that it was decided for protection purposes:

1. That when the proportion of unacceptable batches reached 25 percent there would be a 50 percent chance of an "out of control" signal (that is, it is unlikely (6 percent chance) that more than 4 days will pass before a signal is generated).
2. That when the process average remained at the 1,900-lb level there would be only a 0.15 percent chance of signal below the lower limit.
3. That signals above the upper limit would not be cause for action, but that close attention would be paid to the chart for the succeeding days to determine the extent of the process shift.

Using statistical theory it can be shown that the following relationship must be satisfied:

$$3\,\sigma_{avg} = 3\sqrt{\frac{\sigma^2_{BB}}{n_1} + \frac{\sigma^2_{BS}}{n_1 n_2} + 2\,\frac{\sigma^2_{BR} + \sigma^2_{WR}}{n_1 n_2 n_3}} = 300 \text{ lb}$$

or that

$$\frac{20,000}{n_1} + \frac{20,000}{n_1 n_2} + \frac{79,900}{n_1 n_2 n_3} = \left(\frac{300}{3}\right)^2 = 10,000 \text{ lb.}$$

Therefore, the sampling and testing procedure to be employed will depend on the selection of n_1, n_2 and n_3 such that these values satisfy the foregoing relationship. There are many combinations possible and the aforementioned factors of cost and convenience will determine the final choice.

One set of n's which does satisfy the equation approximately is: $n_1 = 6$, $n_2 = 1$, $n_3 = 4$. In other words, it is necessary to take one sample from each six batches selected randomly, preparing and testing four briquets from each sample. The average for these twelve briquets would be calculated, and plotted on a control chart with center line 1,900 lb and limits set at $1,900 \pm 3\sigma_{ave}$ (i.e., 1,900 ± 300 lb). When an average falls below 1,600 lb (the lower limit) there is a definite indication that the supplier's process has shifted to an unsatisfactory level.

Although this amount of sampling and testing may seem excessive, it is the minimum amount which will provide the required protection, within the limitations of the

Shewhart control chart, and process and testing variability. Only by relaxing the protection desired can this volume of testing be reduced. As mentioned earlier, the practice of making composite samples might lead to some reduction in the work required but the statistical and practical implications would have to be thoroughly examined before a definite conclusion could be made.

A "range" control chart also could be set up to provide some control on the variability of the process, but this will not be described further because the same philosophical concepts apply.

Before leaving this topic, a new type of control chart, the cumulative sum control chart (41, 44) can be briefly discussed.

This form of chart has been used for the last year at the Montreal Research and Control Laboratory and has proven valuable. Its major characteristics are that relatively small shifts of the average become apparent early and it is possible to estimate quickly the size of the shift as well as the date on which the shift occurred. Control limits may be calculated as described in the literature but this was not done, the advantages given being enough to justify use of the charts.

Essentially, the chart is the accumulated total of differences obtained by subtracting the daily average from a fixed value which approximates the process average. Each day this difference (with sign) is added to that of the day before (see Table 13). Shifts in the average show up as changes in the direction of the line, the angle of the line giving a measure of the process average.

TABLE 13

EXAMPLE OF CUMULATIVE SUM CHART CALCULATIONS

Day	$\overline{X}_1$, Avg. of 2 Briquets	$\overline{\overline{X}} - \overline{X}_1$, Difference from 1,900	Cumulative Sum of Differences
1	1,560	- 340	- 340
2	1,850	- 50	- 390
3	1,815	- 85	- 475
4	1,815	- 85	- 560
5	2,085	185	- 375
6	1,700	- 200	- 575
7	2,275	375	- 200
8	1,930	30	- 170
9	2,075	175	5
10	2,150	250	255
11	2,025	125	380
12	2,200	300	680
13	2,100	200	880
14	1,850	- 50	830
15	1,900	0	830
16	2,000	100	930
17	1,585	- 315	615
18	2,050	150	765
19	1,575	- 325	440
20	1,750	- 150	290
21	1,700	- 200	90
22	2,010	110	200
23	1,600	- 300	- 100
24	-	-	-
25	-	-	-
26	-	-	-
-	-	-	-
-	-	-	-

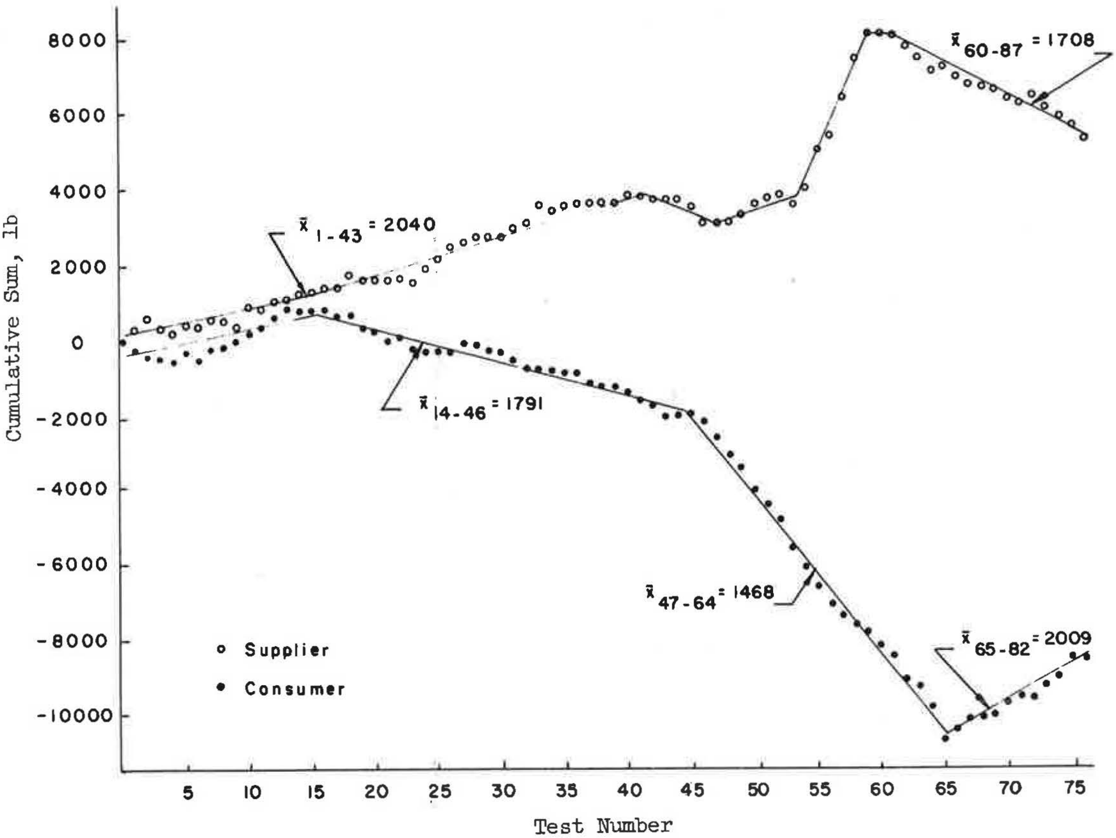


Figure 9. Cumulative sum chart.

Example 4: The process average for a supplier was found to be approximately 1,900 lb. A composite 20-lb sample was prepared from each day's production. Two briquets were made and tested from each composite sample. Calculations were carried out as outlined in Table 12 and the results plotted as in Figure 9.

The cumulative sum chart clearly indicates the following changes:

1. Sequence 1 to 14—Same slope or mean for both the supplier (2,040 lb) and consumer (1,970 lb) test results. (During this period both the supplier's and the consumer's equipment was in good order).

2. Sequence 15 to 46—Same positive slope for the supplier, but a negative slope for the consumer. (During this period, owing to the fact that the process, sampling and testing were unchanged, the change in slope (1,970 to 1,791) was attributed to defective testing equipment. It should be noted that at the time, the City of Montreal Control and Research Laboratory used Shewhart control chart alone. No action was taken because no anomalies appeared and the stability routine test values remained above 1,500 lb, the required minimum value.)

3. Sequence 47 to 64—Action period. (During this period the consumer's stability value dropped to 1,468 level. The Shewhart control chart signaled that the process was out of control. The supplier was asked to take action and a third laboratory was called in to check the apparatus. Finally, it was found that the consumer's compaction hammers were out of order. It is interesting to see, by the several changes in slope, how the supplier actually tried to correct the situation.)

4. Sequence 65 to 82—Finally the consumer corrected the situation by using two new

hammers, the average value (2,009) increased to the original level (2,040). Unfortunately, however, the contractor's equipment was defective (1,708).

Since that time (1960) the City of Montreal has used the cumulative sum chart as part of its control procedure.

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Appendix

TYPICAL DATA COMPILATIONS AND ANALYSES OF VARIANCE

TABLE 14

MARSHALL TEST DATA FOR SURFACE MIX FROM SUPPLIER II, DATA INVESTIGATION

Stone Filled				Stability	Flow	Density
Week	Day	Hammer	Order			
1	1	C	1	1,575	16	2.374
		D	2	1,650	19	2.371
		C	3	1,780	16	2.377
		D	4	1,655	15.5	2.369
	2	C	1	1,715	14.5	2.373
		D	2	1,615	15.5	2.380
		C	3	1,575	17	2.375
		D	4	1,725	16.5	2.378
	3	C	1	1,350	15	2.360
		D	2	1,425	16	2.365
		C	3	1,550	15.5	2.364
		D	4	1,510	16.5	2.366
	4	C	1	1,475	14	2.382
		D	2	1,475	14	2.380
		C	3	1,490	14	2.378
		D	4	1,475	15	2.371
Total				25,040	244.0	37.963
2	1	C	1	1,500	15	2.378
		D	2	1,450	15	2.376
		C	3	1,460	16	2.378
		D	4	1,575	16	2.373
	2	C	1	1,600	17	2.371
		D	2	1,555	15	2.370
		C	3	1,710	16.5	2.371
		D	4	1,750	16	2.370
	3	C	1	1,410	15.5	2.364
		D	2	1,450	15	2.373
		C	3	1,550	14.5	2.364
		D	4	1,600	15	2.365
	4	C	1	1,425	15.5	2.374
		D	2	1,410	15.5	2.365
		C	3	1,645	17	2.379
		D	4	1,575	17	2.373
Total				24,665	251.5	37.944
3	1	C	1	1,650	17	2.377
		D	2	1,460	14.5	2.370
		C	3	1,600	14.5	2.363
		D	4	1,700	15	2.374
	2	C	1	1,595	14	2.367
		D	2	1,535	14.5	2.365
		C	3	1,695	14	2.362
		D	4	1,510	13.5	2.357
	3	C	1	1,600	15.5	2.368
		D	2	1,695	14	2.371
		C	3	1,710	15.5	2.375
		D	4	1,675	16	2.373
	4	C	1	1,600	14.5	2.356
		D	2	1,590	14.5	2.370
		C	3	1,590	15	2.362
		D	4	1,575	16	2.371
Total				25,780	238.0	37.901
4	1	C	1	1,625	14.5	2.365
		D	2	1,500	13.5	2.357
		C	3	1,660	13.5	2.373
		D	4	1,700	14.5	2.362
	2	C	1	1,530	15.5	2.364
		D	2	1,575	14	2.370
		C	3	1,575	13	2.371
		D	4	1,595	12.5	2.368
	3	C	1	1,710	13.5	2.368
		D	2	1,560	13.0	2.362
		C	3	1,840	13.5	2.369
		D	4	1,690	12.5	2.365
	4	C	1	1,555	14.5	2.363
		D	2	1,475	14.5	2.362
		C	3	1,660	13.5	2.366
		D	4	1,655	13	2.354
Total				25,995	219.0	37.339

TABLE 15

ANALYSIS OF VARIANCE[a] FOR MARSHALL STABILITY AND DENSITY, SURFACE MIX, SUPPLIER II, LABORATORY INVESTIGATION

Source of Variation	Degrees of Freedom	Components of Variance	Marshall Stability[c]			Marshall Density[d]		
		Est. by Mean Square[b]	Sum of Squares	Mean Square	F Test	Sum of Squares ($\times 10^{-6}$)	Mean Square ($\times 10^{-6}$)	F Test
Between weeks	3	$16\sigma^2_{BR} + 4\sigma^2_{BD} + 2\sigma^2_{BR} + \sigma^2_{WR}$	72,791	24,264	-	567	189	$F_{(3, 12)}$ 1.8[e]
Between days	12	$4\sigma^2_{BD} + 2\sigma^2_{BR} + \sigma^2_{WR}$	292,798	24,400	$F_{(12, 15)}$ 5.4[f]	1,239	103	$F_{(12, 47)}$ 5.1[f]
Between runs:								
First vs second	1	$32\Sigma f^2_k + 2\sigma^2_{BR} + \sigma^2_{WR}$	108,076	108,076	$F_{(1, 15)}$ 23.7[f]	9	9	$F_{(1, 15)}$[e]
Within day	15	$2\sigma^2_{BR} + \sigma^2_{WR}$	62,281	4,552	$F_{(15, 31)}$ 1.3[e]	259	17	$F_{(15, 31)}$[e]
Between hammer positions	1	$32\Sigma f_l^2 + \sigma^2_{WR}$	4,391	4,391	$F_{(1, 31)}$ 1.3[e]	47	47	$F_{(1, 31)}$ 2.35[e]
Within run	31	σ^2_{WR}	107,238	3,459	-	613	20	-
Total	63		653,575			2,734		

[a]Model: $Y_{ijkl} = \mu + a_i + a_{ij} + a_{ijk} + f_k + f_l + a_{ijkl}$.

[b]BW = between weeks; BD = between days; BR = between runs; WR = within run.

[c]Components of variance: $\hat{\sigma}^2_{WR} = 3,460$, $\hat{\sigma}^2_{BD} = 4,960$, $\hat{\sigma}^2_{BR} = 550$ (NS), $\sigma^2_{BW} = 0$.

[d]Components of variance: $\hat{\sigma}^2_{WR} = 0.000020$, $\hat{\sigma}^2_{BD} = 0.000021$, $\hat{\sigma}^2_{BR} = 0.0$, $\hat{\sigma}^2_{BW} = 0.000005$.

[e]Not significant at 0.05 level.
[f]Significant at 0.01 level.

TABLE 16

ANALYSIS OF VARIANCE[a], FOR RICE'S MAXIMUM DENSITY, SURFACE MIX SUPPLIER II, LABORATORY INVESTIGATION

Source of Variation	Degrees of Freedom	Sum of Squares ($\times 10^{-6}$)	Mean Square ($\times 10^{-6}$)	Components of Variance Est. by Mean Square[b, c]	F Test
Between weeks	3	206	69	$8\sigma^2_{BW} + 2\sigma^2_{BD} + \sigma^2_{WD}$	$F_{(3, 12)}$[d]
Between days, within a week	12	727	61	$2\sigma^2_{BD} + \sigma^2_{WD}$	$F_{(12, 16)}$ 6.8[e]
Within day	16	137	9	σ^2_{WD}	
Total	31	1,070			

[a]Model: $Y_{ijk} = \mu + a_i + a_{ij} + f_j + a_{ijk}$.

[b]Components of variance: $\hat{\sigma}^2_{WD} = 0.00009$, $\hat{\sigma}^2_{BD} = 0.000026$, $\hat{\sigma}^2_{BW} = 0.000001$.

[c]BW = between weeks; BD = between days; WD = within day.

[d]Not significant at 0.05 level.

[e]Significant at 0.01 level.

TABLE 17

STABILITY, FLOW, AND DENSITY DATA OF FIELD SAMPLES, SURFACE MIX, SUPPLIER I

Sequence Day	Marshall Stability				Marshall Flow				Marshall Bulk Density				Rice's Maximum Density	
	Run I		Run II		Run I		Run II		Run I		Run II		Run I	Run II
	X1	X2	X3	X4	X1	X2	X3	X4	X1	X2	X3	X4	X1	X2
1	1700	1700	1750	1850	11.0	10.0	10.0	13.0	2.378	2.397	2.380	2.382	2.453	2.454
2	1715	1875	1900	1815	13.5	15.0	16.0	16.0	2.355	2.360	2.361	2.359	2.434	2.431
3	1685	1625	1585	1700	16.0	15.0	17.0	17.0	2.367	2.366	2.361	2.377	2.423	2.419
4	1550	1510	1650	1555	20.0	19.0	16.0	17.0	2.366	2.355	2.356	2.358	2.414	2.426
5	1605	1575	1590	1525	17.0	17.0	18.0	18.0	2.364	2.365	2.361	2.360	2.419	2.422
6	1400	1450	1500	1490	19.0	22.0	20.0	20.0	2.361	2.364	2.366	2.361	2.401	2.407
7	1490	1660	1810	1780	17.0	17.0	16.0	17.0	2.373	2.371	2.374	2.359	2.415	2.429
8	2005	1830	2040	1900	16.0	14.0	16.0	15.0	2.370	2.374	2.375	2.375	2.450	2.440
9	1600	1535	1645	1490	19.0	15.0	16.0	17.0	2.362	2.365	2.361	2.367	2.417	2.421
10	1925	1950	2005	1900	12.0	12.0	11.0	12.0	2.368	2.369	2.372	2.370	2.447	2.441
11	1460	1600	1475	1450	16.0	16.0	16.0	16.0	2.370	2.373	2.366	2.371	2.416	2.416
12	1435	1285	1265	1485	15.0	17.0	16.0	16.0	2.372	2.368	2.370	2.369	2.402	2.411
13	1445	1550	1470	1470	15.0	13.0	14.0	15.0	2.356	2.357	2.351	2.349	2.410	2.410
14	1470	1550	1550	1635	16.0	15.0	16.0	15.0	2.366	2.371	2.364	2.368	2.421	2.422
15	1610	1460	1445	1535	15.0	15.0	17.0	15.0	2.372	2.361	2.358	2.362	2.416	2.423
16	1710	1910	2010	1970	17.5	17.5	16.0	17.5	2.364	2.379	2.378	2.376	2.424	2.424
17	1850	1520	1635	1710	15.0	17.0	15.0	14.0	2.372	2.365	2.361	2.363	2.433	2.428
18	1685	1710	1710	1970	15.0	15.0	16.0	16.0	2.369	2.371	2.375	2.377	2.424	2.425
19	1850	1760	1785	1635	15.0	15.0	17.0	16.0	2.372	2.370	2.373	2.371	2.433	2.424

TABLE 18

ANALYSIS OF VARIANCE[a] FOR MARSHALL STABILITY AND DENSITY, SURFACE MIX, SUPPLIER I, FIELD INVESTIGATION

Source of Variation	Degrees of Freedom	Marshall Stability[c]: Components of Variance Est. by Mean Square[b]	Sum of Squares	Mean Square	F Test	Marshall Density[d]: Sum of Squares ($\times 10^{-6}$)	Mean Square ($\times 10^{-6}$)	F Test
Between days	18	$4\sigma^2_{BD} + 2\sigma^2_{BR} + \sigma^2_{WR}$	2,076,566	115,364	$F_{(18, 18)}$ 16.5[e]	3,463	192	$F_{(18, 18)}$ 10.7[e]
Between runs:								
First vs second	1	$38\Sigma f^2_j + 2\sigma^2_{BR} + \sigma^2_{WR}$	27,284	27,284	$F_{(1, 18)}$ 3.9[f]	22	22	$F_{(1, 18)}$ 1.2[f]
Within day	18	$2\sigma^2_{BR} + \sigma^2_{WR}$	125,629	6,979	$F_{(18, 38)}$[f]	317	18	$F_{(18, 38)}$[f]
Within run	38	σ^2_{WR}	299,625	7,884		820	22	
Total	75		2,529,104			4,622		

[a]Model: $Y_{ijk} = \mu + a_i + a_{ij} + f_j + a_{ijk}$.
[b]BD = between days; BR = between runs; WR = within run.
[c]Components of variance: $\hat{\sigma}^2_{WR} = 7,900$, $\hat{\sigma}^2_{BR} = 0$, $\hat{\sigma}^2_{BD} = 27,100$.
[d]Components of variance: $\hat{\sigma}^2_{WR} = 0.000022$, $\hat{\sigma}^2_{BR} = 0$, $\hat{\sigma}^2_{BD} = 0.000043$.
[e]Significant at 0.01 level.
[f]Not significant at 0.05 level.

TABLE 19

ANALYSIS OF VARIANCE[a] FOR RICE'S MAXIMUM DENSITY, SURFACE MIX, SUPPLIER I, FIELD INVESTIGATION

Source of Variation	Degrees of Freedom	Sum of Squares ($\times 10^{-6}$)	Mean Square ($\times 10^{-6}$)	Components of Variance Est. by Mean Square[b, c]	F Test
Between days	18	5,806	323	$2\sigma^2_{BD} + \sigma^2_{WD}$	$F_{(18, 19)}$ 15.4[d]
Within day	19	401	21	σ^2_{WD}	
Total	37	6,207			

[a]Model: $Y_{ij} = \mu + a_i + a_{ij}$.
[b]Components of variance: $\hat{\sigma}^2_{WD} = 0.000021$, $\hat{\sigma}^2_{BD} = 0.000150$.
[c]BD = between days; WD = within day.
[d]Significant at 0.01 level.