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Contents

Influence of Mineral Aggregate Structure on Properties of Asphalt Paving Mixtures

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•THE PERFORMANCE of asphalt concrete pavement, a mixture of mineral aggregate, asphalt and air voids, depends largely on the properties and relative proportions of these components. The influence of the aggregate, comprising by far the largest volume or weight proportion of the pavement, is extremely important on such performance. Mineral aggregates may vary widely in their mineralogical, granulometric, strength, surface texture, and shape characteristics. All these properties should be considered when suitability of mineral aggregate is evaluated. This report, however, is concerned primarily with the influence of the geometric factors, or shape and surface texture, on properties of asphalt concrete mixed and compacted by conventional laboratory methods or obtained from the actual pavements.

Visual examination of mineral aggregate, regardless of whether it is a naturally rounded gravel or highly angular crushed rock, invariably reveals that the axial dimensions of individual particles vary. These differences are caused by the presence of weak crystalline planes or soft grains within the aggregate particle. When subjected to mechanical crushing or grinding by natural forces, the aggregate tends to break in the weaker areas, resulting in irregular particles of elongated or flattened shape. In an asphalt paving mixture, the compaction process tends to orient the long axes of aggregate particles in a position perpendicular to the direction of compaction force. This alignment may cause not only different strength and rheological behavior in different directions of the compacted specimens, but also may result in different load transfer properties of the whole pavement layer, depending on the compaction method employed.

Because the anisotropic behavior of compacted asphalt mixtures is affected by the differences in aggregate structure or texture, material specifications often define such characteristics of an aggregate to be used for paving purposes. For example, the number of crushed aggregate faces may be specified and sometimes an attempt is made to limit flattened or elongated particles. U.S. Army Engineer Waterways Experiment Station developed test methods (1, 2) to determine the amount of these particles in coarse and fine aggregates. Although intended for aggregates used in portland cement concrete, these tests also are used by the Corps of Engineers when designing asphalt pavements for military installations. A rather complete description and discussion of methods for measuring particle shape and surface characteristics of mineral aggregates has been prepared by Mather (3). However, a review of the literature indicates a lack of systematic laboratory or field investigations specifically evaluating the effects of particle orientation on the rheological behavior and load-carrying properties of compacted asphalt paving mixtures. Shklarsky and Livneh (4, 5, 6) provided a direct proof for the anisotropic behavior of asphalt mixtures. These authors, using either compression or splitting tests and analyzing theoretically the mechanism of specimen rupture, concluded that the variable coefficient of cohesion in different directions is the reason for this behavior in compacted asphalt mixtures.

It can be expected that the magnitude of the effects of aggregate particle alignment, causing anisotropic physical behavior of compacted mixtures, would be affected not only

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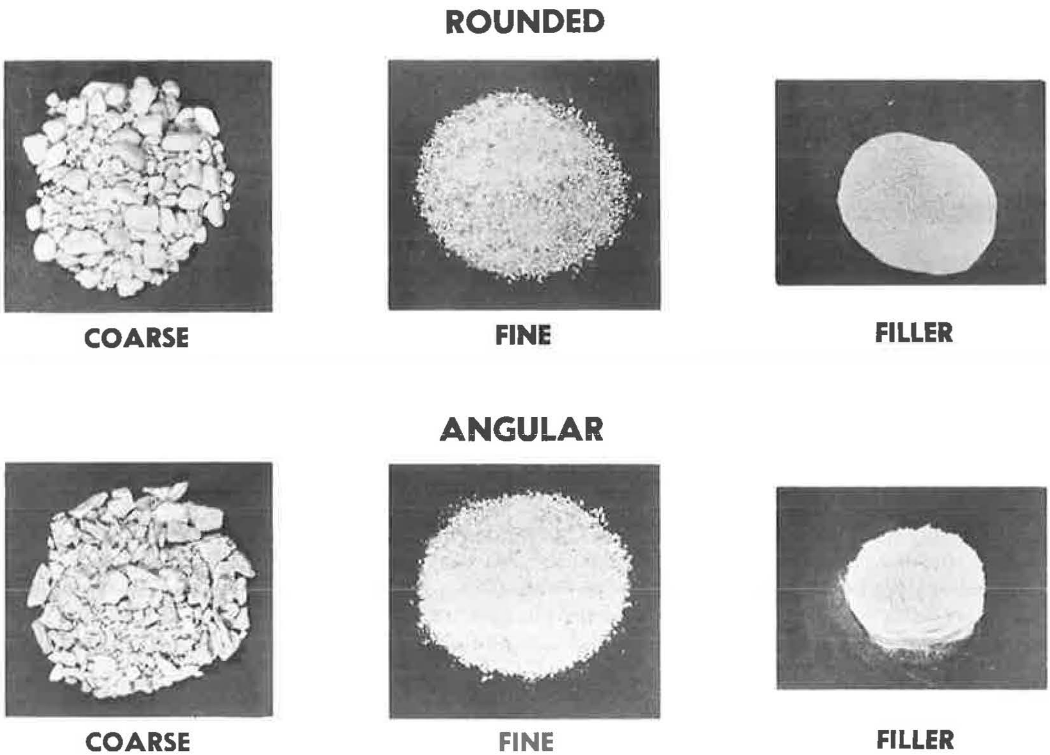

Figure 1. Types of aggregates.

by the type and relative proportions of mixture components, but also by the differences in testing environment. However, in this respect only a very limited amount of information is available in the literature. The purpose of the testing described in addition to evaluating effects on particle alignment of primary factors such as type and relative proportions of mixture components, was to evaluate secondary factors such as variable test temperature, loading rate and type of laboratory compaction. A limited number of tests assessing aggregate particle alignment in field-compacted asphalt mixtures also are described.

DESCRIPTION OF MATERIALS

Mineral Aggregates

Two types of mineral aggregates having the same gradation characteristics were used in this study. One aggregate consisted of rough, angular and flattened or elongated stone fragments and the other contained rounded and smooth-surfaced particles. For angular aggregate, crushed South Carolina granite was used as a coarse fraction while the fine aggregate fraction, including mineral filler, was prepared by crushing locally available Maryland gravel. In the case of rounded aggregate, natural Maryland gravel was used for coarse and fine fractions and Mississippi loess, consisting of rounded particles passing U.S. Standard Sieve No. 200, was used as mineral filler. Figure 1 shows the differences in shape and surface roughness characteristics of coarse and fine aggregate and mineral filler fractions of both rounded and angular aggregates.

Gradation characteristics of these aggregates are shown in Figure 2. The solid line represents the principal aggregate gradation used. Broken lines represent aggregate gradations whose characteristics affect particle alignment in compacted asphalt mixtures. For such supplementary studies, only angular aggregate consisting of crushed South Carolina granite and Maryland gravel mixture was used. The shape of

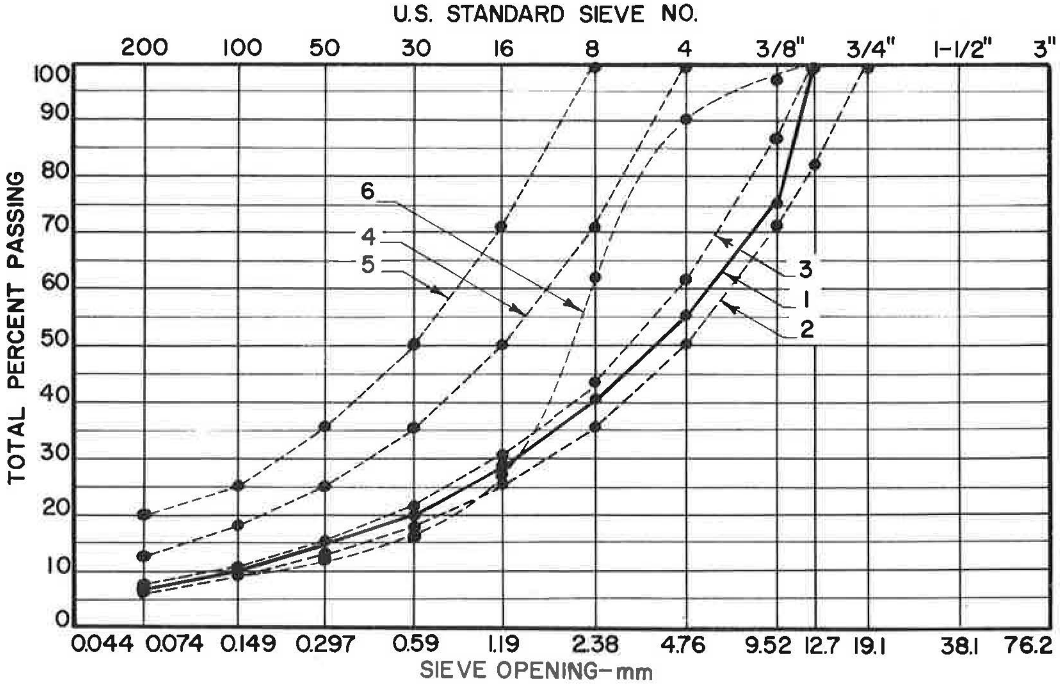

Figure 2. Gradation characteristics of aggregate.

TABLE 1

Asphalt	Pen., 77 F (cm)	Viscosity		Thin Film Oven Test Residue		Viscosity Ratio, 140 F (original: heated)
		140 F (poises x 10^3)	275 F (stokes)	Pen., 77 F (cm)	Viscosity, 140 F (poises x 10^3)	
A	88	2.54	4.49	44	11.43	2.2
B	94	1.04	2.91	54	3.56	3.4
C	61	2.46	4.0	32	7.23	3.0
D	40	4.99	5.7	19	20.37	3.8
E	21	515.0	92.6	16	725.0	1.4
F	0	1080.0	39.5	0	3050.0	2.8

lines in Figure 2 indicates that with the exception of aggregate No. 6, gradation of all other aggregates was selected to follow maximum density curves, sometimes referred to as Fuller's curves. It was believed that gradation of this type would represent a condition less favorable for alignment of the aggregate particles during compaction.

Properties of Asphalt Cements

Table 1 gives the properties of asphalt cements used in this study. Asphalt A was used for the majority of tests. Other asphalts were used only when studying the effects of asphalt viscosity on the aggregate particle alignment in the laboratory-compacted specimens.

EXPERIMENTAL PROCEDURES

Preparation of Mixtures

Accurate aggregate gradation control was achieved by separating all aggregates, in either eight or nine size fractions, by sieving through the series of U. S. Standard Sieves (Fig. 2). Before mixing with asphalt, these fractions were recombined to obtain desired gradation characteristics and the combined aggregate was heated overnight at 325 F in a forced-draft oven. Normally, the asphalt was preheated with intermittent stirring for approximately 2 hr at 275 F before mixing with aggregate. However, asphalts having considerably higher viscosity at 275 F than Asphalt A were heated at higher temperatures so that their viscosity for mixing did not exceed approximately four stokes. All ingredients were mixed for $2\frac{1}{2}$ min with a Hobart laboratory mixer equipped with a 5-qt capacity mixing bowl.

Asphalt contents of 4. 5 and 3. 5 percent were used for angular and rounded aggregate, respectively. These asphalt contents were based on Marshall 50 blow design curves but are approximately one percent lower than the optimum asphalt content as established by this design method.

Compaction of Mixtures

Specimens were compacted by four different compaction methods: (a) Marshall compaction by applying 50 tamps on each face of specimen by compaction hammer, (b) Triaxial Institute Kneading Compactor using 150 tamps at 500-psi pressure on tamping foot and 1,000-psi leveling load, (c) mechanical gyratory compactor using 30 gyrations of 100-psi pressure and 1^{0} angle of gyration, and (d) static load of 3,000 psi applied for 2 min after rodding and tamping mixture for seating in the compaction mold. Molds of 4-in. diameter were used and specimens were compacted to a height slightly larger than 2. 5 in. In some instances certain deviations from standard compaction conditions were necessary to achieve a specific purpose. For example, when evaluating effects of density or air void content on aggregate particle alignment, the number of gyrations with the mechanical gyratory compactor was varied to obtain gradual changes in air void content of compacted specimens.

Preparation of Cube-Shaped Specimens

After cooling at room temperatures, density and air void contents of compacted specimens were determined by weighing specimens in air and in water. All specimens were then cut to cubes of 2. 5-in. side length with a diamond saw. To obtain cubes with opposite faces even and perfectly parallel, the flat surfaces of the cylindrical specimens also were trimmed. Figure 3 shows two 2. 5-in. cubes containing the angular and rounded aggregate particles. Close visual examination of this photograph reveals that particles of both aggregates are aligned in the direction perpendicular to that of

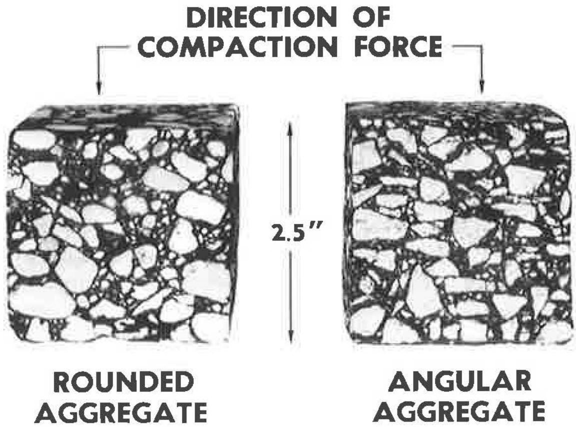

Figure 3. Cubes for compressive strength.

compaction force. However, it can also be seen that such alignment for angular aggregate is considerably more pronounced.

Measurement of Compressive Strength

To assess degree of aggregate particle alignment in compacted mixtures, compressive strengths of cube-shaped specimens were measured in directions normal and parallel to the force of compaction. These measurements were made with a Baldwin Universal Testing Machine by applying loads at a constant rate until the test specimens failed. During loading, stresses and strains on the test specimen were automatically recorded. A rate of 0.05 in./min/in. of specimen height was selected as standard. However, a number of tests using different load application rates also were performed.

During testing the temperature was held constant to within ± 0.2 F by placing specimens in a heavy gage, flat-bottom steel container and pumping water through this container from a thermostatically controlled water bath. Normally, test specimens were kept in the water bath for at least 1 hr before testing.

Measurement of Aggregate Particle Alignment

A ratio of compressive strengths, "Aggregate Structure Index," was employed as a measure indicating degree of aggregate particle alignment or aggregate structure in the compacted mixtures. This index is calculated by dividing the compressive strength of cubic specimens measured in the direction parallel to the compaction by that measured in the direction perpendicular to compaction. If the numerical value for the structure index equals unity, the aggregate particles within the compacted mixture are distributed at random, and not axially aligned in either direction. Values greater than unity indicate that aggregate particles are aligned or stacked perpendicular to the compaction force, and that such stacking causes higher strengths when cubic specimens are tested in the direction parallel to the force of compaction. Structure index values lower than unity indicate that the long axes of particles are aligned parallel to compaction force. However, regardless of aggregate type or test environment, structure index values were always greater than unity. Greater deviations of this index from unity indicated either more pronounced alignment of aggregate particles or more pronounced effects of such alignment on directional strengths of compacted mixtures.

TEST RESULTS

In general, agreement between test values measured on individual specimens was relatively good, provided the aggregate for each specimen was obtained from the same batch. For example, densities as measured on individual specimens differed from the average value by less than ½ pcf and differences in air void contents did not exceed ½ of 1 percent. Variations in compressive strength between duplicate specimens seldom exceeded 10 percent of the average value. If such variation in compressive strengths exceeded 15 percent, new specimens normally were compacted and the experiment was repeated.

However, if aggregate from another batch were used, differences between test values of specimens prepared from the two aggregate batches usually were greater than the differences between specimens prepared from the same batch. Such variation normally was more pronounced with the crushed-angular aggregate than with rounded gravel because of the difficulty in control and reproduction of aggregate angularity when crushing and processing the stock material at different times.

Densities and air void contents were obtained on cylindrical specimens before cutting to cubical shape. Tests indicate that densities (unity weights) of specimens containing both aggregates were normally higher and air void contents lower for the cut, cubic specimens than for the cylindrical specimens before cutting because the metal walls of the compaction mold tend to hinder the densification of mixture. Therefore, cutting and removal of the less dense mixture in the proximity of the compaction mold wall results in a greater density of the remaining test cube.

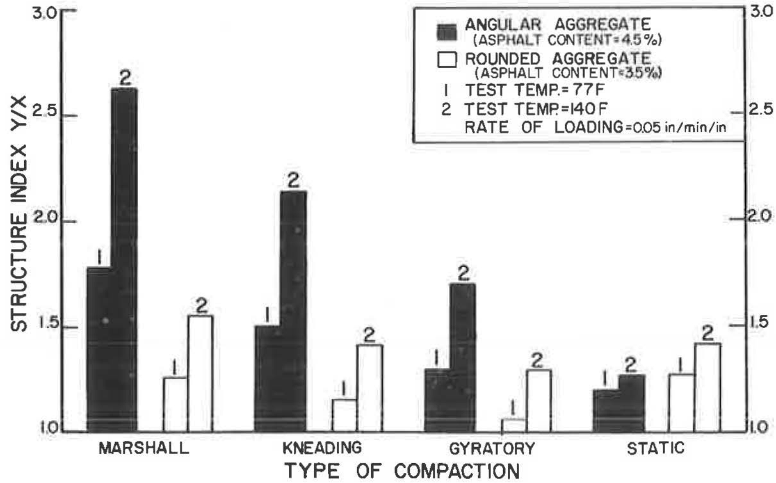

Figure 4. Effect of type of compaction on alignment of angular and rounded aggregate particles.

TABLE 2

INFLUENCE OF AGGREGATE TYPE AND COMPACTION METHOD ON PARTICLE ALIGNMENT AT TWO TEST TEMPERATURES

Type of Compaction	Aggregate Type	Asphalt Content[1] (%)	Unit Weight[2] (pcf)	Void Cont.[2] (vol. %)	Test Temp. (°F)	Compr. Strength[3] Y Direct. (psi)	Compr. Strength[3] X Direct. (psi)	Structure Index (Y/X)
Marshall,	Angular	4.5	146.3	5.2	77	498	281	1.77
50 blows on each	Angular	4.5	146.4	5.1	140	207	78	2.63
specimen face	Rounded	3.5	150.7	4.8	77	313	247	1.26
	Rounded	3.5	150.6	4.5	140	85	54	1.56
Triaxial Institute	Angular	4.5	147.2	4.6	77	520	345	1.51
Kneading Compactor,	Angular	4.5	147.8	4.3	140	212	101	2.13
500 psi, 150 tamps	Rounded	3.5	153.3	3.0	77	470	405	1.16
	Rounded	3.5	153.3	3.0	140	132	94	1.41
Gyratory compactor,	Angular	4.5	148.0	4.0	77	371	258	1.30
100 psi, 1° angle,	Angular	4.5	147.8	4.2	140	189	111	1.70
30 gyrations	Rounded	3.5	151.2	4.3	77	323	304	1.06
	Rounded	3.5	151.5	4.0	140	77	61	1.27
Static compaction,	Angular	4.5	144.5	6.5	77	505	422	1.20
3,000 psi	Angular	4.5	144.7	6.4	140	87	69	1.26
	Rounded	3.5	147.5	6.6	77	467	371	1.26
	Rounded	3.5	147.6	6.5	140	62	44	1.41

[1] Asphalt A used for specimen preparation. [2] Average values of four specimens.
[3] Rate of loading 0.05 in./min/in. of specimen height. Average values of two specimens.

DISCUSSION OF TEST RESULTS

Influence of Type of Compaction on Alignment of Angular and Rounded Aggregate Particles

Figure 4 shows effects of four compaction methods on particle alignment of angular and rounded particles, as indicated by the Aggregate Structure Index. Compaction methods included in Figure 4 represent dynamic-impact (Marshall), kneading (Triaxial Kneading Compactor), and gyratory and static compaction. For mixtures containing angular aggregate, 4.5 percent of Asphalt A was used and for mixtures with rounded aggregate, 3.5 percent was used. Compressive strengths in two directions of cubic specimens were measured at 77 and 140 F using a standard loading rate of 0.05 in./min/in. of specimen height. Complete test properties for these mixtures are given in Table 2.

Figure 4 indicates that regardless of aggregate type and method of compaction, aggregate particles when subjected to compactive forces always tend to align themselves perpendicular to force of compaction. As indicated by greater than unity Index values, such alignment always results in higher strengths when measured by loading parallel to the force of compaction (Y direction) than perpendicular to that force (X direction).

Although Figure 4 shows that both shape of aggregate particles and type of compaction greatly influence alignment of aggregate particles, the effect of the former usually is somewhat more pronounced. With the exception of static compaction, comparison of structure indices for mixtures compacted by any other method reveals that at both 77 and 140 F these indices for angular aggregate are more than double those for rounded aggregate mixtures. In static compaction, at both test temperatures the structure index for rounded aggregate is somewhat larger than for mixtures with angular aggregate. This is probably caused by high friction between rough and angular surfaces that hinders densification when compacted under confining static loads. Such resistance is markedly lower with rounded and smooth-surfaced aggregate particles. However, when the intermittent impact-type loads are applied, as with Marshall or kneading compaction, the influence of friction appears to be considerably less and the alignment of the elongated or flattened particles more pronounced.

Figure 4 shows that for angular aggregate mixtures, the intermittent impact (Marshall type) compaction represents the most favorable condition for particle alignment. The next is kneading compaction which also can be classified as an intermittent impact condition, although of a different type. This is followed by the gyratory and, finally, by the static compaction types. The two latter methods may be described as confined and continuous loading compaction methods which, apparently, allow considerably less chance for aggregate particle movement and, therefore, axial alignment. For mixtures with rounded aggregate, similar trends are indicated by Figure 4. However, static compaction again is an exception to this general trend.

Figure 4 also indicates that differences among structure indices for rounded aggregate mixtures compacted by the four methods are far less pronounced than for mixtures with angular aggregate. This implies that the strength, and possibly the other properties, of rounded aggregate mixtures is considerably less influenced by the laboratory compaction method than similar properties of mixtures containing rough-surfaced and irregularly shaped aggregate particles. In regard to compaction of actual pavements this may mean that for paving mixtures containing naturally rounded gravels, the selection of the type of compaction equipment for efficient mixture densification may be less important than for the mixtures containing difficult to compact, rough and angular crushed stone.

Further, differences in the structure index for different compaction methods (Fig. 4) imply that the correlations between test values as obtained by currently used laboratory mixture design methods should not be expected. This is true for the Marshall method, because Marshall stabilitites are measured by compressing the specimen in a direction other than the direction of the force applied during the compaction process.

The variability in structure index values also raises questions concerning the precision of laboratory design methods for asphalt paving mixtures. Often, stability test results as measured on the individual specimens vary considerably even when prepared by the same operator using identical asphalt-aggregate mixtures, particularly those containing crushed, large maximum size aggregate particles. The slight differences in the initial placement or seating procedures of mixtures in compaction molds may affect the alignment of the aggregate particles during the subsequent compaction process resulting in pronounced differences between stability values for the individual specimens.

Effect of Asphalt Content on Aggregate Particle Alignment

Data shown in Figure 4 and Table 2 represent tests made at a single asphalt content for each type of aggregate mixture. To evaluate the influence of asphalt content on particle alignment, standard Marshall design tests, and determination of structure indices, were made at different asphalt contents (Figs. 5, 6, Table 3). For these tests, Asphalt A was mixed at different proportions with both rounded and angular aggregates having the same gradation characteristics, as represented by the solid line in Figure 2.

Figure 5 shows changes in unit weights and air void contents for both rounded and angular aggregates when mixed and compacted with variable amounts of asphalt. Data presented indicate that the compaction characteristics of rounded and angular aggregates differ appreciably, regardless of the fact that both aggregates have the same gradation

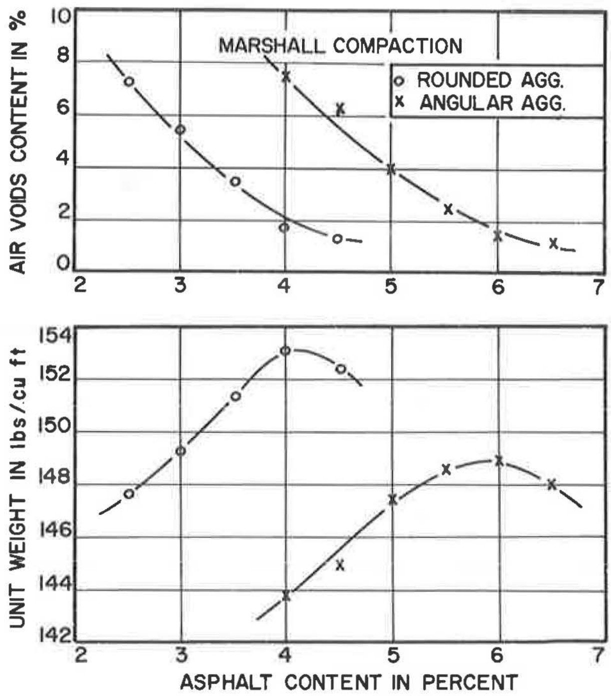

Figure 5. Compaction characteristics of rounded and angular aggregate mixtures.

TABLE 3

EFFECT OF ASPHALT CONTENT ON ALIGNMENT OF ROUNDED AND ANGULAR AGGREGATE PARTICLES IN PAVING MIXTURES

Aggregate Type	Asphalt Content[1] (%)	Unit Weight[2] (pcf)	Air Void Content[2] (vol. %)	Compr. Strength[3] Y Direct. (psi)	Compr. Strength[3] X Direct. (psi)	Structure Index (Y/X)	Marshall Stability (lb)
Rounded	2.5	147.6	7.2	64	47	1.37	1,170
	3.0	149.2	5.5	82	65	1.25	1,360
	3.5	151.3	3.5	99	71	1.40	1,290
	4.0	153.0	1.6	102	72	1.42	1,210
	4.5	152.3	1.4	90	60	1.50	918
Angular	4.0	143.8	7.6	216	64	3.39	1,332
	4.5	144.9	6.2	219	78	2.80	1,390
	5.0	147.3	3.9	198	83	2.39	1,432
	5.5	148.6	2.4	176	84	2.10	1,462
	6.0	148.9	1.5	162	82	1.98	1,420
	6.5	148.0	1.4	146	60	2.43	1,260

[1] Asphalt A used for specimen preparation.
[2] Marshall compaction, 50 tamps on each end of specimen. Average values of six specimens.
[3] Test temperature 140 F; loading rate 0.05 in./min/in. of specimen height. Average values of two specimens.

characteristics. At the same asphalt contents, rounded aggregate mixtures compact to a considerably higher density and lower air void content than do similar mixtures with angular aggregates. In addition, asphalt contents for maximum densities for mixtures containing rounded aggregate are appreciably lower than for mixtures containing angular aggregate.

Figure 6 illustrates the relationship of asphalt content to directional compressive strengths and structure indices for mixtures with rounded and angular aggregates. Regardless of the asphalt content and type of aggregate, compressive strengths as measured in the Y direction are always higher than those measured in the X direction. For the rounded aggregate mixtures, however, differences in directional strengths are small and appear to be nearly constant over the whole range of asphalt contents, resulting in a relatively small change in the structure index with changing asphalt content.

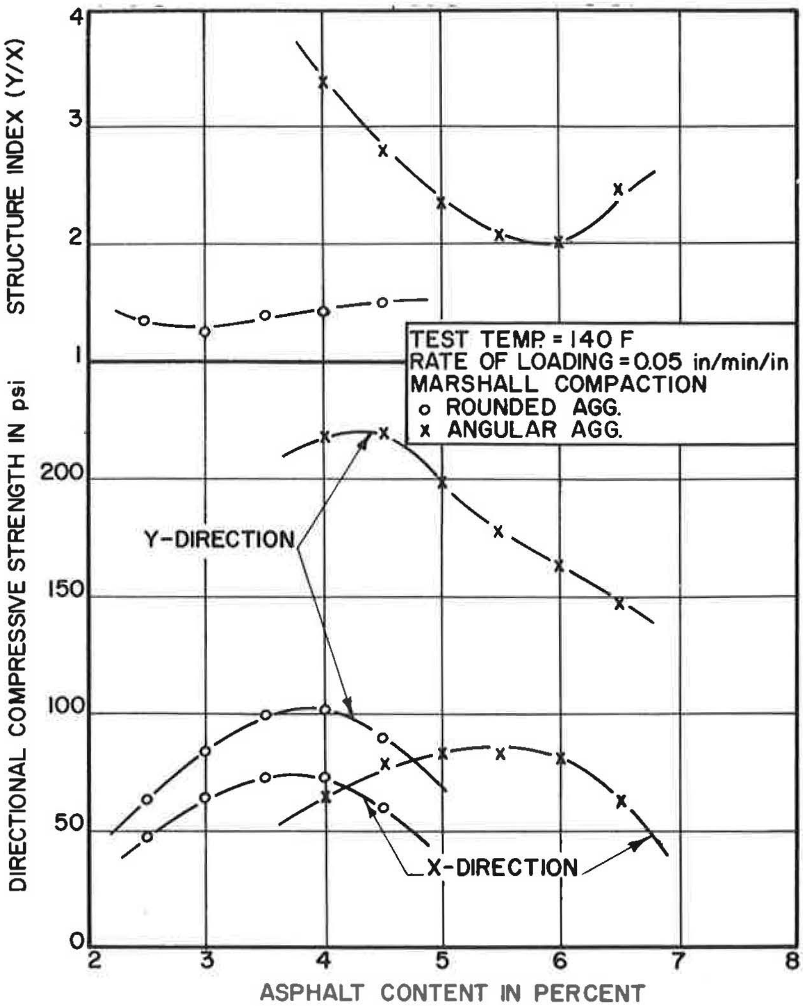

Figure 6. Effect of asphalt content on structure index of mixtures of rounded and angular aggregates.

Differences between directional compressive strengths with angular aggregate are by far more pronounced, varying with asphalt content. For example, at asphalt content of 4 percent, compressive strength in the Y direction is nearly four times as great as in the X direction. At 6 percent asphalt content, however, this difference is reduced by a factor of 2. Thus, as shown in the upper part of Figure 6, the structure index for mixtures with angular aggregate is much more strongly influenced by the variations in asphalt content than is the similar index for rounded aggregate mixtures.

Marshall stabilities, as far as mixture strength properties are concerned, do not provide similar information. Test results in Table 2 show that the maximum values are approximately the same for angular and rounded aggregates. The variation of Marshall stability with varying asphalt content is more similar to the variation of compressive strengths when measured in X direction. This is not surprising because Marshall stabilities are also measured by applying compressive loads in the same direction.

Relationship Between Air Void Content and Alignment of Aggregate Particles

The relationships between air void content, directional compressive strengths, and

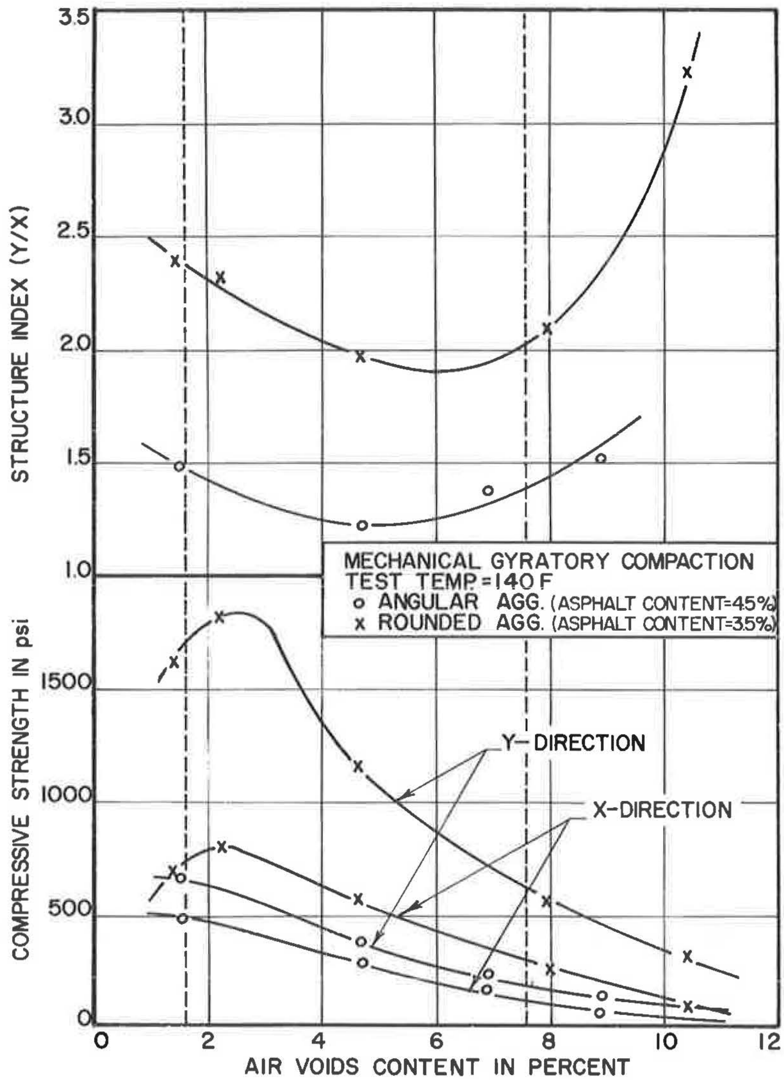

Figure 7. Effect of air void contents on alignment of rounded and angular aggregate.

aggregate particle alignment are illustrated in Figure 7. Rounded and angular aggregate mixtures were compacted to different air void contents with the mechanical gyratory compactor. Air void volumes were controlled between approximately 2 and 10 percent by continually measuring specimen height during compaction. For these tests, constant asphalt contents of 4.5 and 3.5 percent were used for angular and rounded aggregates, respectively. Compressive strength determinations in both directions were carried out at 140 F, using a loading rate of 0.05 in./min/in. of specimen height.

The lower part of Figure 7 shows that regardless of aggregate type or air void content, compressive strength measured parallel to compaction force is always higher than the strength perpendicular to such a force. Such differences are far more pronounced for angular than for rounded aggregates. Furthermore, the compressive strengths in both testing directions increase with decreasing air void contents. For angular aggregate, compressive strengths appear to reach a maximum value at approximately 2 percent air voids. For rounded aggregate, however, such trends are not clearly established.

The upper part of Figure 7 indicates that at air void contents of 6 percent for angular aggregate and 5 percent for rounded aggregate, the structure indices reach minimum values of 1.9 and 1.2, respectively. Below and above such air void contents they tend to be higher. Again, as in the case of compressive strengths, changes in structure index with changing air void contents for the angular aggregate are by far more pronounced than for mixtures containing rounded aggregate.

In general, influence of air void contents on the structure index or particle alignment is considerably less than the influence of factors such as type of aggregate, type of compaction or asphalt content. This is especially true within the range of air voids between approximately 2 and 8 volume percent as indicated by the vertical broken lines in Figure 7. This range represents variation in air voids of mixtures containing both aggregates, and compacted either by different compaction methods or at different asphalt contents (Figs. 4, 5 and 6). Figure 7 indicates that within this air void content range variations in structure index are relatively small, implying that effects of air void content on aggregate particle alignment (Figs. 4 and 6) generally were not great.

Effect of Rate of Loading on Structure Index

Test data discussed so far were concerned with compressive strengths measured at a single, constant rate of loading. Figure 8 represents compression tests at variable rates of loading. In this figure, test results only with angular aggregate mixtures compacted by the Marshall method at 4.5 percent asphalt content are shown. Directional compressive strength tests were performed at 77 and 140 F.

The shapes of both lines in Figure 8 indicate that effects of rate of loading on structure

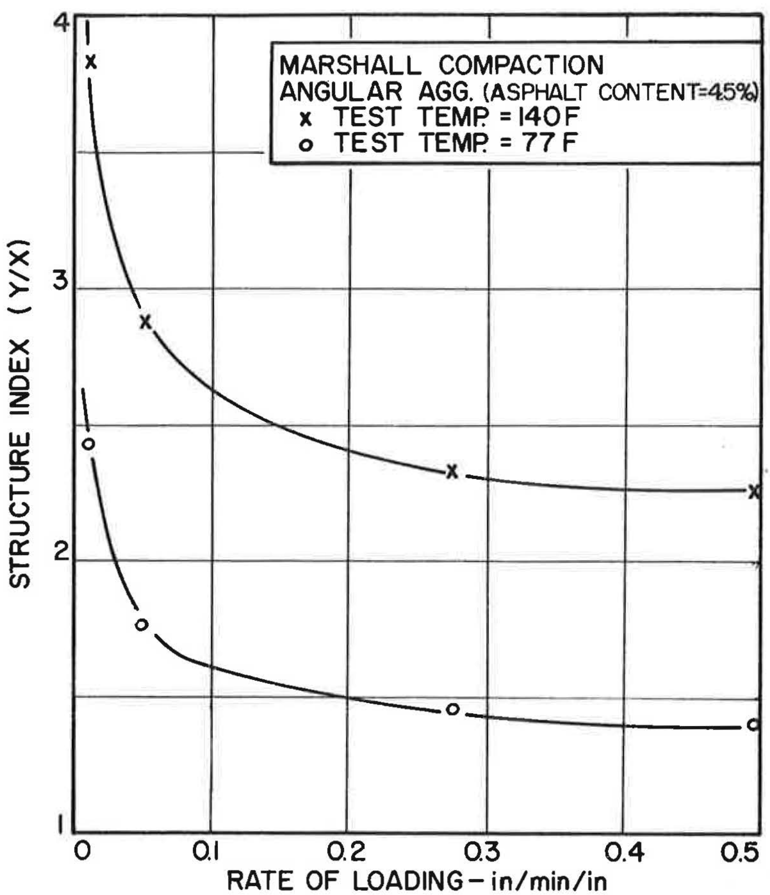

Figure 8. Effect of loading rate on ratio of directional compressive strength.

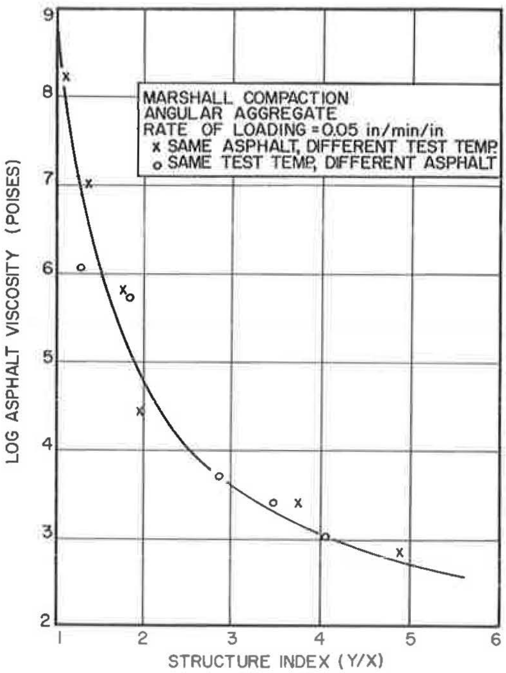

Figure 9. Effect of asphalt viscosity or test temperature on directional compressive strengths.

index are quite similar for both test temperatures. Structure index values tend to increase with decreasing rates of loading. At high rates of loading such increases are relatively small. However, at the rates of loadings close to 0.05 in./min these lines inflect, and increases in structure index with decreasing rate of loading become very pronounced. Generally compressive strengths in both directions increase with increasing rate of loading. However, at low rates of loading, increases of compressive strength in the Y direction are far more pronounced. This, then, causes the inflection point in structure index vs loading rate lines shown in Figure 8. This inflection point coincides approximately with rate of loading selected as standard for tests described in this report and with that required in a number of ASTM or AASHO tests for compressive strength measurements.

Effect of Asphalt Viscosity and Test Temperature on Structure Index of Paving Mixtures

The influence of asphalt viscosity or test temperature on structure index of mixtures containing angular aggregate is illustrated in Figure 9 and data were obtained by two different methods. Test values represented by crosses were obtained on mixtures containing Asphalt A only. Viscosities of this asphalt and directional compressive strength of mixtures were measured at 39.2, 60, 80, 100, 140 and 160 F. Test data represented by open circles were obtained on mixtures containing asphalts of different viscosity from compressive strength tests at 140 F. (Viscosity values at 140 F for different asphalts are given in the section on Description of Materials.) For all these tests, mixtures were prepared at an asphalt content of 4.5 percent with standard Marshall compaction and standard rate of loading were used.

Figure 9 shows that structure index is strongly dependent on either the test temperature or the asphalt viscosity and that effects of both of these factors are very similar. With the decreasing viscosity these indices increase. For example, at an asphalt viscosity of about one billion poises, commonly encountered near freezing temperatures, the structure index value is close to unity. At viscosities of approximately one thousand poises, representing pavement temperatures near the maximum reached in service, the structure index is about four for these angular aggregates. The value of unity for structure index means that strengths of mixture in all directions are equal. A structure index value of 4 means that the strength in Y direction is four times greater than that in X direction. Thus, the strength properties of mixtures at freezing temperatures are predominantly dependent on the viscosity of asphalt binder. However, as the viscosity of asphalt decreases or temperature of pavement increases, the influence of aggregate characteristics gradually becomes greater, until at close to 140 F they become the most important factor determining the properties and behavior of paving mixtures.

Influence of Aggregate Gradation on Structure of Paving Mixtures

Figure 10 illustrates effects of different aggregate gradations and maximum aggregate particle sizes on alignment of these particles when subjected to compactive forces. For these tests, mixtures of only the angular aggregate and Asphalt A were used. With the exception of gradation No. 6, particle size distributions shown in Figure 2 represent

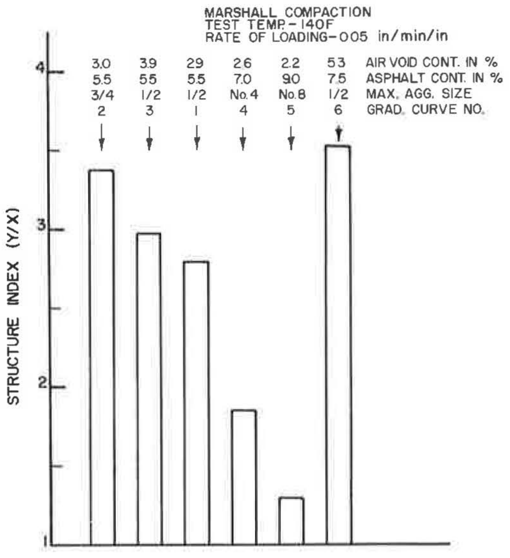

Figure 10. Effect of aggregate gradation on particle alignment.

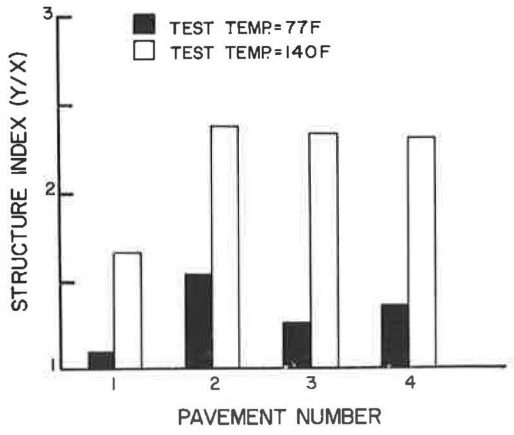

Figure 11. Alignment of aggregate particles in actual pavements.

maximum density (Fuller's) curves for a given maximum aggregate size. Aggregate No. 6 represents the so-called "skip-graded" aggregate that is deficient in coarse and fine sizes. Test cubes cut from cylindrical Marshall specimens were tested only at 140 F by applying compression loads in two directions at the standard loading rate. These mixtures were compacted to approximately the same air void content requiring changes in asphalt contents for mixtures with finer graded aggregates. Asphalt and air void content values are shown above each bar representing the different aggregate gradations.

Particle alignment becomes less pronounced with finer graded asphalt paving mixtures. For example, structure index for aggregate gradation No. 2 (maximum particle size ¾ in.) is approximately 3.5. For the finest graded aggregate No. 5 (all passing U.S. Standard sieve No. 8), this index is approximately 1.3. In the case of fine-graded mixtures, such as sand or sheet asphalt, relative differences in axial particle dimensions are smaller and, therefore, the effects of alignment of such particles on strength characteristics of compacted mixtures become less pronounced. Therefore, it may be assumed that asphalt mastics, mixtures of filler and fine sand with relatively large amounts of asphalt, would behave almost entirely in an isotropic manner.

The particle alignment is the most pronounced for mixtures containing skip-graded aggregate, because such uniformly graded mixtures, lacking in the intermediate particle sizes, contain fewer contact points between aggregate particles.

Aggregate Particle Alignment in Field Compacted Mixtures

For assessment of aggregate particle alignment in actual pavements, directional compressive strengths of cubes cut from pavement cores were measured at 77 and 140 F using standard loading rates. Such evaluations were performed on the following pavements: (a) Virginia experimental pavement after 4 yr of service, (b) North Carolina filler test section 1 mo after completion of construction, (c) base course of Northeastern Expressway in Maryland approximately 1 wk after placement and compaction, (d) test section for pavement temperature measurements at Asphalt Institute Headquarters 2 wk after construction.

The alignment of aggregate particles as reflected by the structure indices at two test temperatures for all four pavements is illustrated in Figure 11. Other properties of these paving mixtures are summarized in Table 4. In all these pavements either the entire aggregate, or at least the coarse aggregate fraction, consisted of crushed and angular particles. Additionally, in all four cases, the maximum aggregate size, asphalt content and air void content also varied.

TABLE 4

ALIGNMENT OF AGGREGATE PARTICLES IN ACTUAL PAVEMENTS

Pavement	Aggregate Type			Asphalt Cont.[1] (%)	Unit Weight (pcf)	Void Cont. (vol. %)	Test Temp. (°F)	Compr. Strength		Structure Index (Y/X)
	Coarse	Fine	Filler					Y Direct. (psi)	X Direct. (psi)	
Virginia exper. pavement	Crushed limestone			6.0	-	-	77	359	327	1.10
				6.0	-	-	140	241	145	1.66
North Carolina filler test section	Crushed limestone	Quartz	Kaolin	6.2	144.0	5.8	77	367	240	1.54
				6.2	144.0	5.8	140	195	82	2.37
Maryland Northeastern Expressway (base)	Maryland trap rock	Sand		4.4	142.4	9.2	77	264	214	1.25
				4.4	144.4	7.9	140	30	13	2.31
Test section at Asphalt Institute	Crushed limestone	Sand		4.9	141.7	9.7	78	254	188	1.35
				4.9	143.4	8.4	140	18	8	2.29

[1] Asphalt of 85-100 penetration grade used in all pavements.

In all four pavements, aggregate particles tended to align with the long axis perpendicular to the compaction force. Such alignment appears to be nearly equal in all but the Virginia test road. This road was subjected to traffic for a considerably longer time than the other three pavements. During this service time, viscosity of asphalt increased appreciably and, as indicated in Figure 9, such increases result in lower structure index values.

Although Figure 11 shows that all four pavements contained aligned aggregate particles, comparison of the alignments between different pavements may not be entirely justifiable because the thickness of pavements and, therefore, size of test cubes cut from different pavements varied. Furthermore, the ratio between maximum aggregate particle size and test cube side length also was variable. Preliminary tests for this study indicated, however, that for accurate compressive strength determination, side length of the test cube must exceed the maximum aggregate size by a factor of 3. This was not always the case with test cubes cut from in-service pavements.

SUMMARY AND CONCLUSIONS

The purposes of the tests described in this report were (a) to assess aggregate particle alignment in compacted asphalt paving mixtures, and (b) to evaluate the influence of such alignment on properties of these mixtures. For these evaluations, the compressive strength of cubic specimens in two different directions was determined and the ratio of the directional strengths (structure index) was employed as a measure indicating the alignments of aggregate particles.

Test data collected in this study lead to the following tentative conclusions:

1. Visual observation and larger than unity values for the structure index indicate that, regardless of the type of aggregate or the method of compaction, aggregate particles in compacted asphalt concrete tend to become axially aligned in a direction perpendicular to the direction of the compaction force.
2. Greater ratios of the directional compressive strength (structure index) indicate that the effects of the particle alignment on the properties of mixtures containing elongated or flattened aggregates tend to be more pronounced than for mixtures containing rounded and smooth-textured aggregates.
3. The alignment of aggregates strongly depends on the method of compaction. Generally, compaction methods utilizing intermittent, impact-type compactive forces represent a more favorable condition for particle alignment than those methods employing continuous and confining compaction forces.
4. Because of differences in aggregate particle alignment, comparisons between test results obtained by different laboratory design methods are difficult and may not be valid. This may be particularly true for mixtures containing crushed and angular aggregates.
5. The effects of asphalt and air void contents on the ratio of directional compressive strength are only moderate, particularly within the ranges of asphalt and air void contents commonly encountered in compacted asphalt paving mixtures.

6. The influence of asphalt viscosity or test temperature on the ratios of directional compressive strength is quite pronounced. With increasing viscosity or decreasing temperature these ratios decrease and approach unity. This appears to indicate that at temperatures near freezing, the strength properties of a mixture depend more on the characteristics of the asphalt. At higher pavement temperatures, however, the characteristics of aggregate become predominant.

7. Regardless of test temperature, directional strength ratios decrease with an increasing rate of load application. However, such changes at a loading rate of 0.05 in./min/in. of specimen height and higher become increasingly insignificant.

8. Alignment of particles tends to increase with increasing size of aggregate particles. For paving mixtures such as sand or sheet asphalt, the structural index tends to approach unity.

9. Test data indicate an appreciable degree of aggregate particle alignment for in-service pavements.

10. Systematic evaluations of the effects of aggregate type and alignment on load transfer properties and performance of in-service pavements are needed. Such evaluations may lead to the more rational use of different types of aggregates and improved pavements.

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Discussion

W. H. CAMPEN and L. G. ERICKSON, Omaha Testing Laboratories.—Mr. Puzinauskas' paper interests us because it deals with practical aspects of the design problem which are not only easily understood but also very timely.

We wish to make two comments concerning Mr. Puzinauskas' paper. The first pertains to aggregate alignment. Conclusions 3 and 9 taken together indicate that intermittent impact methods of compaction produce aggregate alignment more similar to that produced by the action of traffic than do other methods. Because of this, one wonders why the advocates of the use of the gyratory method of compaction reached the conclusion that this method comes nearer to duplicating aggregate orientation in the pavement than any other method. Perhaps Mr. Puzinauskas can elaborate on this point.

The other comment concerns the gradation of Aggregates 1 and 3 (Fig. 2). Mr. Puzinauskas states that they are dense-graded. We wish to disagree with him and say that they are more nearly in the region of open-graded mixtures.

In 1940 (7), we presented a paper to the Association of Asphalt Paving Technologists

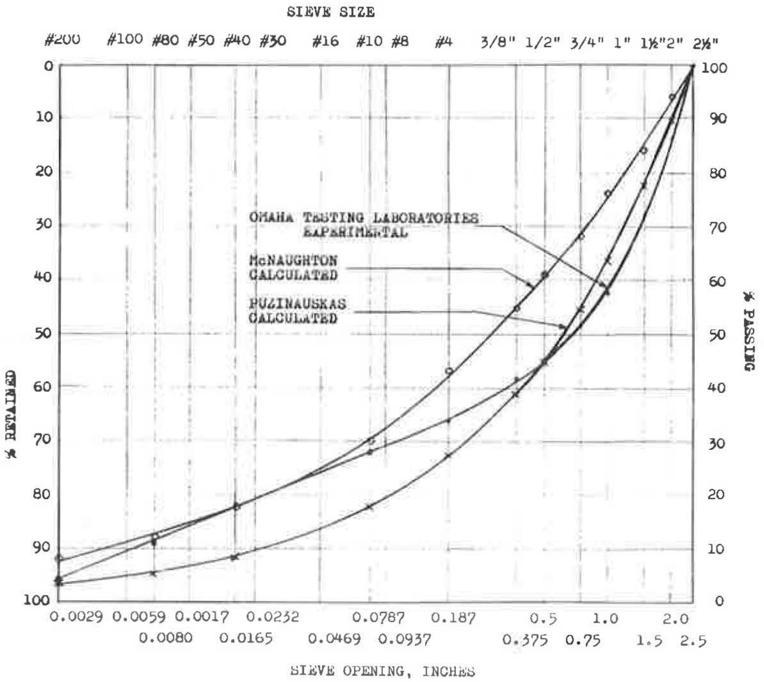

Figure 12. Gradation curves, $2\frac{1}{2}$ in. maximum size.

entitled "The Development of Maximum Density Curve . . . ." The curve (Fig. 12) was developed by preparing 11 sizes of aggregate starting with the No. 200 and ending with the $2\frac{1}{2}$ in. sieves. The various sizes were combined in a systematic manner and compacted by vibration. Figure 12 also shows a curve which we calculated from Mr. Puzinauskas' formula, and a curve submitted by T. F. Macnaughton in a discussion of our paper, which he based on research by C. C. Furnas and W. G. Weymouth.

Figure 13 shows five curves for $\frac{1}{2}$ in. maximum aggregates. The three solid curves were calculated from curves in Figure 12. One of the dashed curves is Puzinauskas' gradation 1 and the other represents a gradation that we have adopted after observing field performance for many years.

A study of the curves indicates in the case of Figure 12 that: (a) the Puzinauskas and OTL curves are similar at greater than $\frac{1}{2}$ in. but the former is much coarser at less than $\frac{1}{2}$ in.; (b) the Macnaughton and OTL curves are similar at less than No. 10 sieve but the latter is much finer at greater than No. 10 sieve; (c) the Puzinauskas curve shows a coarser gradation than the Macnaughton curve throughout. Figure 13 indicates that: (a) both the Puzinauskas curves show coarser gradations than both the Macnaughton and OTL experimental curves, and Puzinauskas curve No. 1 shows coarser gradation than his curve No. 3; (b) the Macnaughton curve shows coarser grading generally than the OTL experimental curve; (c) the OTL typical design curve is similar to the Macnaughton curve at greater than No. 10 sieve but is coarser at less than the same sieve.

The data as a whole in Figure 13 show that the Puzinauskas' gradations are much coarser than the one presented by Mr. Macnaughton or the ones developed and derived by us. Therefore, it is our opinion that the former are open-graded.

In conclusion we wish to add that our remarks are not intended to detract from the main purpose of Mr. Puzinauskas' paper which he accomplished very well. Instead they are made for the purpose of keeping the record straight in regard to dense and open-graded mixtures.

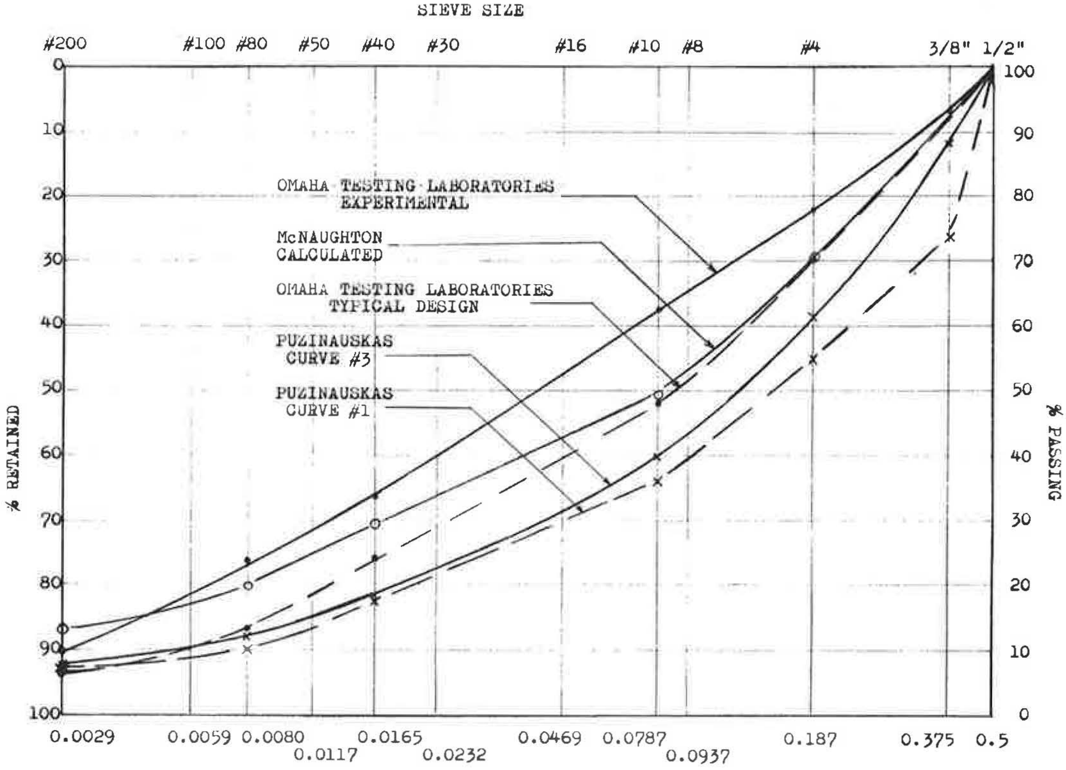

Figure 13. Gradation curves, $\frac{1}{2}$ in. maximum size.

Reference

7. Campen, W. H., and Erickson, L. G., "The Development of a Maximum Density Curve and Its Application to the Grading of Aggregates for Bituminous Mixtures." Proc. Assoc. of Asphalt Paving Technologist, 11:302.

V. P. PUZINAUSKAS, Closure. —The principal purpose of this paper is to illustrate and evaluate the effects of aggregate structure or shape on the properties and behavior of compacted paving mixtures. Only the first of the two comments by Messrs. Campen and Erickson is directly related to this purpose.

In regard to this first comment, it is rather difficult to agree with the discussers' interpretation of conclusions 3 and 9, even when these conclusions are taken out of the context of the paper. These two conclusions do not state, nor do they imply, that some laboratory compaction methods are better than others in reproducing aggregate alignment under the action of traffic.

The writer is also unaware of any published papers stating that the gyratory compaction in laboratory duplicates aggregate particle orientation in the actual pavement better than the other laboratory compaction methods. If such a statement does exist, it would be rather difficult to agree with it because compaction methods differ not only in the laboratory but also in the field. Furthermore, such duplication of aggregate particle orientation would depend on a number of additional factors, such as the type of aggregate, the method of initial placement of the mixture, temperature of compaction and consistency and relative cooling rate of the mixture.

The second comment is a source of puzzlement. The author could not find in the text of the paper a statement implying that aggregates Nos. 1 and 3 are dense-graded.

In the section of the paper describing mineral aggregates it is stated that all aggregates except No. 6 follow maximum density curves, sometimes referred to as Fuller's curves. This, of course, is not the same as stating that these aggregates are dense-graded.

It is interesting to note, however, that according to The Asphalt Institute criteria (i.e., percentage of aggregate passing U.S. Standard Sieve No. 8) aggregates Nos. 1 and 3 could be classified as dense-graded aggregates.

Use of a Gyratory Testing Machine In Evaluating Bituminous Mixtures

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Respectively, Graduate Assistant; and Professor, School of Civil Engineering, and Research Engineer, Joint Highway Research Project, Purdue University

A study was made of selected laboratory test properties of bituminous mixtures compacted by the gyratory testing machine. This compaction was imposed in an attempt to simulate compaction by construction equipment and traffic. The study consisted, in part, of stability and unit weight measurement. Two aggregate gradations were used to study the effects of compaction on dense- and open-graded mixtures. Compactive effort was varied by changing ram pressure and number of revolutions, as well as type of operation in the gyratory testing machine. Statistical analyses were used to evaluate the effect of the machine variables on specimen stability.

The stabilities of specimens compacted in the gyratory testing machine were compared to those compacted in the kneading compactor according to the Hveem design procedure. Stabilities at the same percent voids were analyzed for both kneading- and gyratory-compacted specimens. Compacted specimens were cut in half and unit weight determinations were made for specimen tops and bottoms. Statistical tests were performed to determine whether unit weight gradients existed in compacted specimens.

The results of the study indicated that the load imposed on specimens during the course of the stabilometer test increased density and decreased voids in some compacted specimens. For the more dense mixture, increased initial compaction decreased the secondary compaction that could be applied before loss in stability occurred. For these mixtures, good correlation was noted between the stability values of kneading-compacted and gyratory- compacted specimens for the same percent voids. For the open-graded mixture, gyratory-compacted specimens had higher values of stability than did kneading-compacted specimens for the same percent voids.

Statistical analyses indicated that for the open-graded mixture, the tops of kneading-compacted specimens had appreciably higher unit weights than the bottoms. This relationship was not dependent on asphalt content. For gyratory-compacted specimens, unit weights of specimen bottoms tended to be slightly greater than those of specimen tops in cases where a difference in unit weight was noted.

Gyratory compaction of long slender, aggregate pieces dispersed in a plastic clay medium allowed reorientation of the aggregate so that the long axis of the aggregate pieces tended to be horizontal.

Paper sponsored by Committee on Mechanical Properties of Bituminous Paving Mixtures.

•BITUMINOUS MIX design methods in current use relate design to amount and type of traffic the pavement will be required to withstand (14). Because of varying and unknown traffic loads to which a bituminous pavement is subjected, design and construction criteria may have to be altered occasionally to provide a realistic correlation between laboratory design and in-service traffic conditions. The stability required for pavement at a signalized intersection on a primary truck route may be quite different from that required for a lightly traveled secondary highway. Rutting and shoving of bituminous resurfacing, particularly at signalized intersections (5), indicate that some bituminous mixtures are unstable in certain instances when present design methods predict the mixture should be stable. Evidently design methods in current use are not completely adequate.

Currently used laboratory compaction methods (1, 6) have not been able to reproduce the in-service density of some bituminous mixtures without producing excessive degradation. Type of compaction has been shown to be important to the strength that may be expected from a bituminous mixture (1, 2). Researchers (4, 12, 16, 17, 19, 20) in bituminous mixture design methods have indicated a need for reproducing in laboratory test specimens the same properties that the pavement will acquire when used by traffic.

There are disadvantages of prohibitive cost, unknown or uncontrollable variables, etc., which may hinder useful results of field testing (24) and cause gaps between laboratory designs and in-service traffic condition (8). Horizontal forces due to movement of the tire might be a main cause of difference between field and laboratory compaction (16). Also, laboratory procedures that achieve a given density without regard to aggregate orientation or degradation cannot produce representative specimens (15).

Field research in measuring pavement densification under traffic has found (13) that densification of a mix is proportional to the opportunity a mix has to densify. Soils investigations (26) have shown that steel-wheeled rollers produce the greatest density in a zone close to the roller surface. The Hveem design procedure (21) utilizes the kneading compactor to reproduce degradation and kneading effects similar to those occurring under traffic. Schmidt et al. (25) shows that for excessive compaction with steel-wheeled rollers, pavement density increases with depth from the pavement surface.

The gyratory shear method approximates the in-service pavement condition more closely than other compaction methods (20). In developing improved procedures for design and control of hot-mix bituminous pavements, the U. S. Army Engineer Waterways Experiment Station built a gyratory testing machine based on the compaction method used by the Texas Highway Department (17, 20, 22). This machine was used extensively in correlatory work with pavements subjected to high tire contact pressures (1, 11). It has also been used for density control of highway bituminous paving projects (9). Because the gyratory testing machine produces in laboratory specimens density and stability values approaching those that result from heavy aircraft traffic (1), it was studied for possible applicability to the simulation of highway construction and traffic effects on bituminous mixtures.

To relate the gyratory compaction procedure to one in current use, selected properties of gyratory-compacted specimens were compared with those of kneading-compacted specimens. Because mixture stability was considered one of the most important properties desired in a bituminous mixture, all machine variables were evaluated by their effect on this property.

EQUIPMENT AND TESTING METHOD

Equipment used in this study for the compaction and testing of bituminous mixtures is standardized equipment with the exception of the gyratory testing machine (Figs. 1, 2), which is a mechanized compaction and testing apparatus similar in principle to the manually operated Texas compaction apparatus. Compaction of a specimen occurs when the machine exerts a combined kneading and shearing action on a specimen contained in a steel mold. Vertical pressures are maintained against the specimen by hydraulically controlled steel rams whose faces are parallel to one another. The chuck holding the steel mold is mechanized so that it can move with the revolution of two

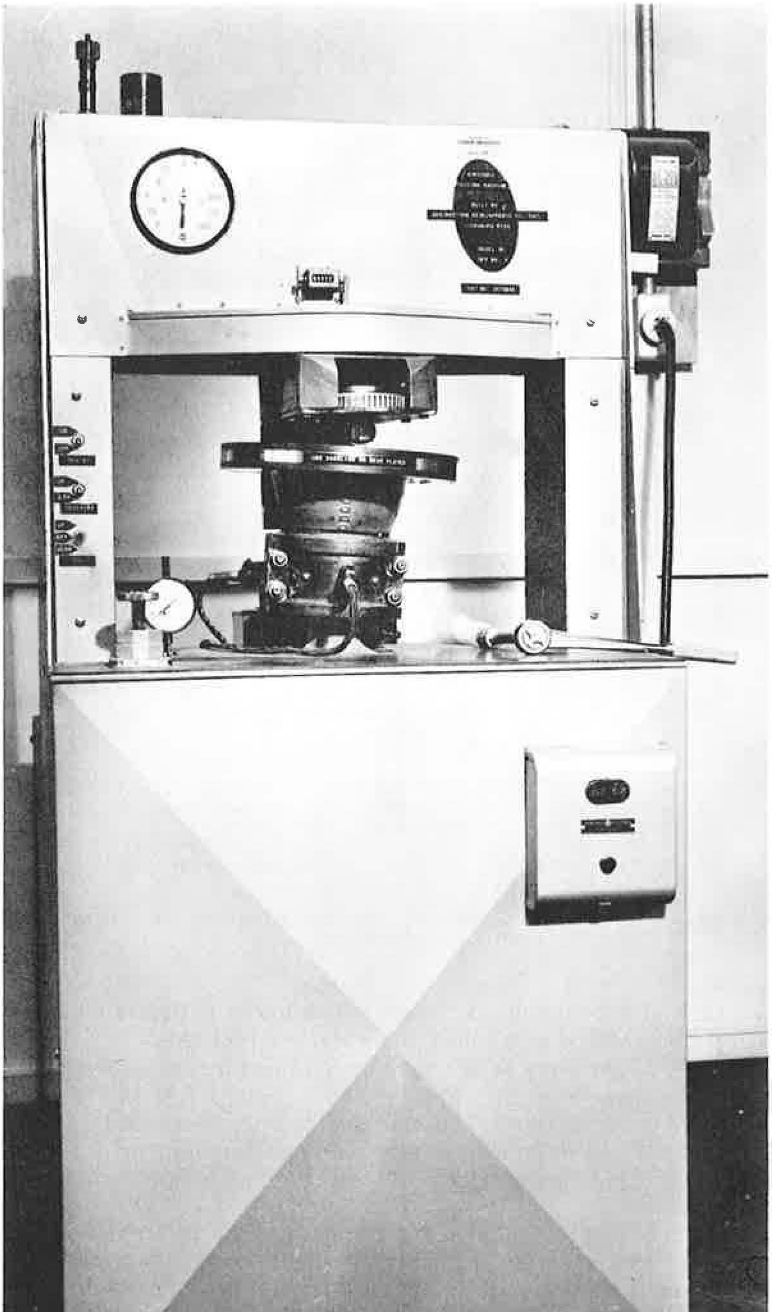
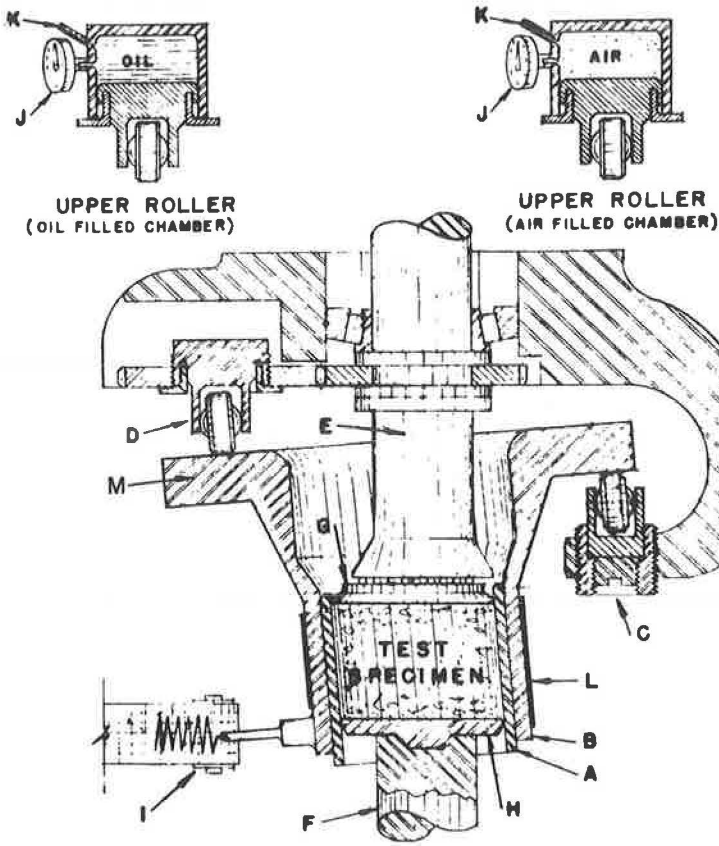


Figure 1. Gyratory testing machine.



A. Specimen Mold
B. Mold Chuck
C. Lower Roller
D. Upper Roller
E. Upper Ram Shaft
F. Lower Ram Shaft
G. Upper Head
H. Lower Head
I. Gyrograph
J. Pressure Gage
K. Filling Valve
L. Heating Element
M. Chuck Flange

Figure 2. Schematic side view of section through gyrating mechanism (after Corps of Engineers).

rollers, one on each side of the chuck flange. The lower roller is adjustable and permits the chuck flange to be rotated or pitched about its vertical axis.

Different degrees of gyratory action may be obtained by employing fixed, air-filled, or oil-filled upper rollers (Fig. 2). Most of the compaction in this study was accomplished using a fixed upper roller. The machine as operated with this roller produces gyratory action of the fixed-deformation type. A smaller number of tests were performed using the air-filled upper roller. This permits a fixed-stress variable-deformation gyratory action.

Although the pitch of the flange on a line connecting the rollers (which act as point loads 180^0 apart) is fixed, the flange can rotate about the line between these two points, and therefore, the mold chuck can develop gyratory angles greater than the angle made by this line. Changes in the gyratory angle reflect the plastic properties of the material in the mold and are recorded on a gyrograph by a mechanical pen recorder. The more plastic and the weaker the specimen, the larger will be the gyratory angle and the wider will be the gyrograph.

The gyratory testing machine used in this study produced a compacted specimen with dimensions compatible for stability testing by several currently used design procedures (14). The Hveem stabilometer (3, 14, 21) was selected as the basis for stability evaluation of compacted specimens because Hveem stability values have had good cor-

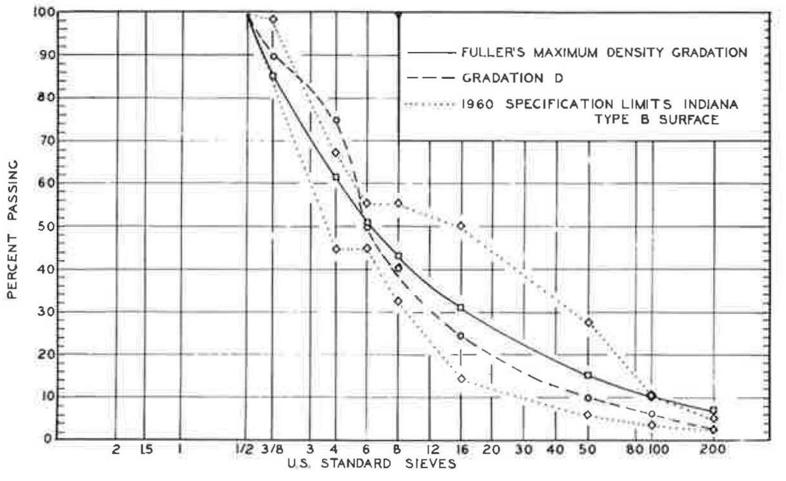


Figure 3. Aggregate gradation curves.

TABLE 1

RESULTS OF TESTS ON AGGREGATES

Material	Size	Spec. Gravity		Absorption (%)
		Bulk	Apparent	
Limestone	1/2 - 3/8 in.	2.63	2.68	1.10
Limestone	3/8 - No. 4	2.67	2.71	0.90
Limestone	No. 4-6	2.63	2.71	1.74
Limestone	No. 6-8	2.62	2.70	1.94
Natural Sand	No. 8-16	2.59	2.72	2.77
Natural Sand	No. 16-50	2.60	2.70	2.45
Natural Sand	No. 50-100	2.63	2.70	2.63
Dune Sand	No. 100-200	2.59	2.65	1.27
Limestone	Passing No. 200	2.71	—	—

relation with field performance of bituminous mixtures. Hveem and Davis (7) believe that materials with varying stabilities do not undergo any marked difference in relative classification whether tested in the stabilometer or in a triaxial device where a theoretical stress analysis is possible. The stabilometer test is relatively fast and easy to perform and, hence, applicable to testing adequately a large number of specimens in a short time.

MATERIALS

The bituminous mixtures used for this study were selected according to specifications of the Indiana State Highway Commission for Hot Asphaltic Concrete Surface—Type B (10). It was thought that the gradations selected would make them applicable to testing in both the gyratory machine and the Hveem stabilometer without special modification of standard test procedures.

The types of aggregates used were crushed limestone, dune sand, natural sand and limestone filler. Aggregate materials were tested for specific gravity and absorption according to ASTM Methods C 127 and C 128. The results of these tests are given in

Table 1. The commercially produced and washed aggregates were sieved into the required sizes and then washed again before storage prior to blending.

The two gradations used in this study are shown in Figure 3. The Fuller's maximum density gradation utilized a ½-in. maximum sieve size and was calculated from the Fuller and Thompson empirical formula

$$P_i = P_o(D_i/D_o)^{1/2} \qquad (1)$$

in which

P_i = percent smaller than D_i

P_o = percent smaller than D_o

D_o = maximum sieve size in gradation

D_i = intermediate sieve size in gradation

Gradation D material was similar to the Type B surface.

Results of tests on the 60 to 70 penetration grade asphalt used are presented in Table 2.

TABLE 2

RESULTS OF TESTS ON ASPHALT CEMENT

Spec. Grav. at 77 F	1.036
Softening Point, Ring and Ball (F)	124
Ductility at 77 F, 5 cm/min (cm)	200 +
Penetration, 100 gm, 5 sec at 77 F	66
Penetration, 200 gm, 60 sec at 32 F	17
Loss on Heating, 50 gm, 5 hr at 325 F (%)	0.01
Penetration of Residue (% of original)	89
Flash Point, Cleveland Open Cup (F)	595
Solubility in CCl_4 (%)	99.84

PROCEDURE

Aggregates separated into component sieve-size fractions were batched according to the blend formula. Aggregate batches of 1100 gm each were used throughout the study and batching was accomplished with cold dried aggregates using a scale sensitive to 1 gm. Prior to mixing, aggregate batches and asphalt were heated separately to 325 ± 5 F. Mixing bowl, paddle, and other utensils were also heated to 325 ± 5 F to minimize heat loss during mixing. Asphalt content for the entire study was specified as percent by weight of the aggregate. The constituents of each batch were mixed in a modified Hobart mixer for 2 min and then transferred to curing pans for a 15 hr period at 140 ± 5 F in a Hotpack oven provided with forced draft air circulation. After the curing period, each batch was reheated to 225 ± 5 F for compaction.

Two types of compaction were used in this study: kneading and gyratory. Kneading compaction was performed with the California kneading compactor using the procedure outlined by the Asphalt Institute (14).

The sequence of compaction in the gyratory testing machine was chosen to simulate compaction that might be expected from construction equipment and traffic. Accordingly, compaction in the gyratory testing machine was divided into two phases—initial compaction and secondary compaction. Initial compaction was carried out with the fixed upper roller and a specimen temperature of 225 ± 5 F. Either 10 or 20 initial compaction revolutions were imposed on the specimen in an attempt to bracket the range of compaction a bituminous layer might receive from construction compaction equipment. Ram pressures of 50, 100, and 150 psi were utilized.

Secondary compaction involved 30, 60, 90, or 400 additional revolutions at secondary pressures of 50, 100, or 150 psi and a temperature of 140 F. These pressures were selected to simulate normally severe tire contact pressures imposed by traffic.

After completion of compaction in either the gyratory testing machine or the California kneading compactor, specimens were tested in the Hveem stabilometer by the procedure outlined by the Asphalt Institute (14).

Bulk specific gravity determinations were made for all compacted specimens after stabilometer testing. Rice specific gravity was obtained for those specimens for which percent voids were to be computed. The procedure is detailed in ASTM Special Technical Publication No. 191 (23).

Uniformity of unit weight with specimen height was studied by cutting the compacted specimens in half with a masonry saw. Because the sawing operation wetted the speci-

TABLE 3

COMPARISON OF BULK UNIT WEIGHT BEFORE AND AFTER STABILOMETER TEST

Bulk Unit Weight (pcf)		
Before	After	Difference[1]
142.0	144.1	2.1
141.6	143.5	2.1
141.0	142.9	1.9
141.0	142.9	1.9
142.9	144.1	1.2
143.5	144.8	1.3
141.6	143.5	1.9
143.5	144.8	1.3
141.6	142.9	1.3
141.0	142.9	1.9
Total		16.7

[1]Mean difference = $\frac{16.7}{10}$ = 1.67 pcf.

men halves, they were washed free of dust and placed in water for a 24-hr absorption period. After the submerged and saturated surface-dry specimen weights were recorded, the specimen halves were placed on absorbent paper and air dried at room temperature for 24 hr. The weight in air was then recorded and specific gravity determinations were made.

RESULTS

Influence of Stabilometer Test on Compacted Specimens

Specimens tested in the Hveem stabilometer were deformed in the course of testing. To evaluate the effect of this deformation on the unit weight of compacted specimens, ten specimens were compacted in the gyratory testing machine using a 100 psi ram pressure, 10 rev., and a 1^0 angle of gyration. Bulk unit weights of these specimens were determined after compaction and after testing in the stabilometer and it was found that the stabilometer test caused a significant increase in these weights (Table 3).

Fixed-Roller Operation

To investigate the variation in stability caused by the factors involved in the gyratory compaction process an analysis of variance test was used. A series of three-way analysis variables included initial pressure, secondary pressure, and secondary revolutions. Two gradations, dense and open, were used to study effects of aggregate gradation and asphalt content was not varied. A total of 192 specimens containing 4 percent asphalt were tested by the procedures outlined previously, with the exception that only 30, 10, or 90 secondary revolutions were employed.

Four three-way analysis of variance tests were required to analyze data common to each of two gradations and two values of initial revolutions. A ranking of the relative

TABLE 4

ANALYSIS OF VARIANCE (FIXED EFFECTS MODEL)[a]

Factor[b]	Degrees of Freedom	Sum of Squares	Mean Squares	Variance Ratio	$F_{0.05}$	Decision[c]
A	2	58.39	29.20	7.19	3.89	Reject H_0
B	3	1198.76	399.59	98.42	3.49	Reject H_0
C	2	132.03	66.02	16.26	3.89	Reject H_0
AB	6	10.18	1.70	0.42	3.00	Accept H_0
AC	4	19.24	4.81	1.18	3.26	Accept H_0
BC	6	94.84	15.81	3.89	3.00	Reject H_0
ABC	12	48.74	4.06	—	—	—

[a]Fuller's maximum density gradation, 10 rev. initial compaction.

[b]A = secondary revolutions, 3 levels; B = secondary pressure, 4 levels; C = initial pressure, 3 levels.

[c]H_0: Stability not affected by factor (α = 0.05).

TABLE 5

ANALYSIS OF VARIANCE (FIXED EFFECTS MODEL)[a]

Factor[b]	Degrees of Freedom	Sum of Squares	Mean Squares	Variance Ratio	$F_{0.05}$	Decision[c]
A	2	75.95	37.98	15.76	3.89	Reject H_0
B	3	1197.34	399.11	165.61	3.49	Reject H_0
C	2	222.19	111.10	46.10	3.89	Reject H_0
AB	6	21.14	3.52	1.46	3.00	Accept H_0
AC	4	8.93	2.23	0.93	3.26	Accept H_0
BC	6	91.64	15.27	6.34	3.00	Reject H_0
ABC	12	28.95	2.41	—	—	—

[a]Fuller's maximum density gradation, 20 rev. initial compaction.
[b]A = secondary revolutions, 3 levels; B = secondary pressure, 4 levels; C = initial pressure, 3 levels.
[c]H_0: Stability not affected by factor (α = 0.05).

TABLE 6

ANALYSIS OF VARIANCE (FIXED EFFECTS MODEL)[a]

Factor[b]	Degrees of Freedom	Sum of Squares	Mean Squares	Variance Ratio	$F_{0.05}$	Decision[c]
A	2	193.54	96.77	76.20	3.89	Reject H_0
B	3	1879.22	626.41	493.24	3.49	Reject H_0
C	2	160.97	80.49	63.38	3.89	Reject H_0
AB	6	74.77	12.46	9.81	3.00	Reject H_0
AC	4	5.79	1.45	1.14	3.26	Accept H_0
BC	6	6.68	1.11	0.87	3.00	Accept H_0
ABC	12	15.28	1.27	—	—	—

[a]Gradation D, 10 rev. initial compaction.
[b]A = secondary revolutions, 3 levels; B = secondary pressure, 4 levels; C = initial pressure, 3 levels.
[c]H_0: Stability not affected by factor (α = 0.05).

TABLE 7

ANALYSIS OF VARIANCE (FIXED EFFECTS MODEL)[a]

Factor[b]	Degrees of Freedom	Sum of Squares	Mean Squares	Variance Ratio	$F_{0.05}$	Decision[c]
A	2	180.95	90.48	127.44	3.89	Reject H_0
B	3	1424.03	474.68	668.56	3.49	Reject H_0
C	2	310.16	155.08	218.42	3.89	Reject H_0
AB	6	45.08	7.51	10.58	3.00	Reject H_0
AC	4	1.46	0.37	0.52	3.26	Accept H_0
BC	6	19.03	3.17	4.46	3.00	Reject H_0
ABC	12	8.52	0.71	—	—	—

[a]Gradation D, 20 rev. initial compaction.
[b]A = secondary revolutions, 3 levels; B = secondary pressure, 4 levels; C = initial pressure, 3 levels.
[c]H_0: Stability not affected by factor (α = 0.05).

TABLE 8

ANALYSIS OF VARIANCE (FIXED EFFECTS MODEL)
5-WAY CLASSIFICATION

Factors	Mean Sum of Squares	Degrees of Freedom	F	$F_{60, 0.5}$	Decision[a]	Estimate of σ^2 Factor
Secondary revolutions	387.8	3	129.3	2.76	Reject H_0	8.0
Secondary pressures	279.2	3	93.1	2.76	Reject H_0	5.7
Initial pressures	475.8	2	237.9	3.15	Reject H_0	7.4
Initial revolutions	324.7	1	234.7	4.00	Reject H_0	2.4
Gradations	80.2	1	80.2	4.00	Reject H_0	0.8

[a]H_0: Stability not affected by factor (α = 0.05).

importance of the three factors in effecting changes in stability can be obtained from the size of the mean squares given in the last columns of Tables 4, 5, 6, and 7. Generally, for the increments chosen, the factors most important in increasing stability values were, in order of importance: secondary pressure, initial pressure, and secondary revolutions.

A five-way analysis of variance test (18) was then made (Table 8). It differed from the three-way analysis of variance in that it included also the factors of initial revolutions, gradations, and 400 secondary revolutions. A quantitative estimate of the importance of each factor may be obtained from the relative sizes of the numbers listed in the column headed "Estimate of σ^2 Factor." For this analysis, the factors most significant in changing specimen stability were, in order of importance: secondary revolutions, initial pressure, secondary pressure, initial revolutions, and gradation.

The ranking of factors in the five-way analysis of variance differed from the three-factor ranking most noticeably in the reversal of the importance of secondary pressure

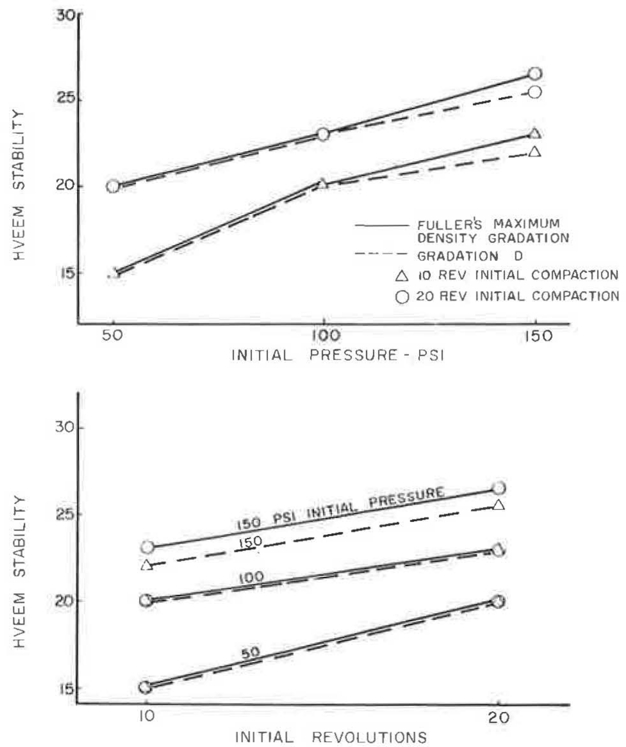


Figure 4. Effects of initial pressure and initial revolutions on specimen stability.

and secondary revolutions caused by the large (400) secondary revolutions value added to the levels of this factor. A controlled field study is necessary to determine how closely field compaction was simulated by the sequences of laboratory compaction.

The analysis of variance technique used here is a general method that may be used for investigating the effects of any number of variables on specimen properties. The estimate of σ^2 factor shown in the last column of Table 8 may be replaced in a more comprehensive study by estimates of regression for each factor. In this way linear, quadratic, and higher order effects of each factor could be measured. A computer analysis would be necessary for large-scale correlation between laboratory and field results.

Initial Compaction. —The effect of initial compaction pressure on initial stability of specimens is shown in a plot of Hveem stability vs initial pressure (Fig. 4). For this portion of the study a constant asphalt content of 4 percent was used. Increasing the

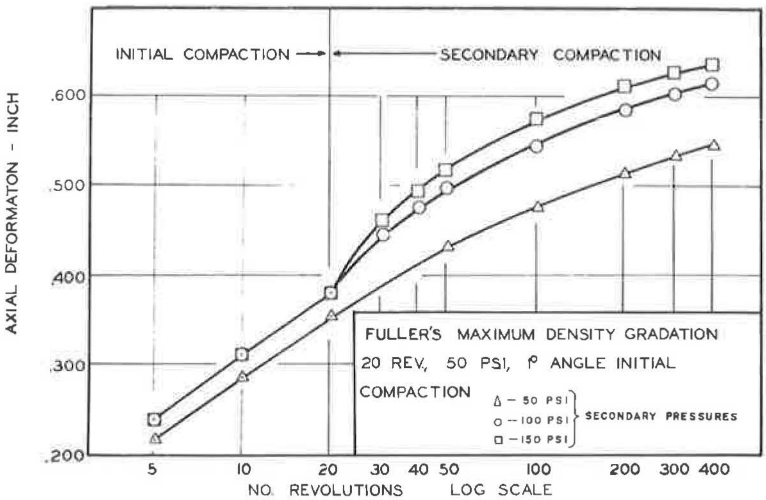


Figure 5. Axial deformation vs no. of revolutions, 50 psi initial compaction.

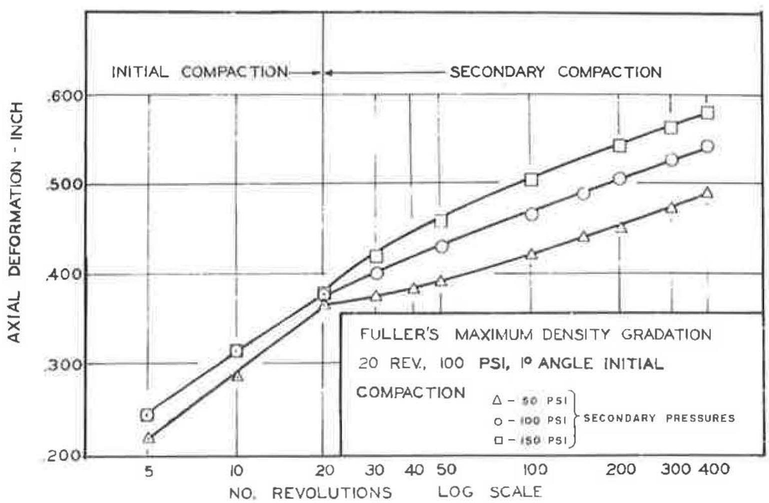


Figure 6. Axial deformation vs no. of revolutions, 100 psi initial compaction.

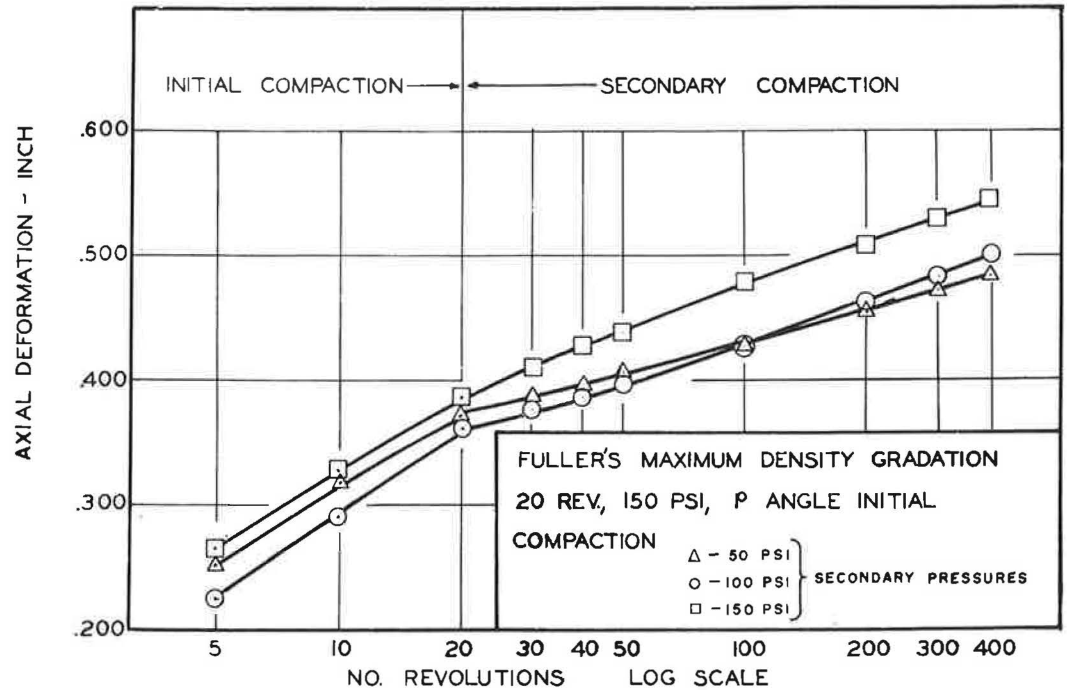


Figure 7. Axial deformation vs no. of revolutions, 150 psi initial compaction.

initial compaction pressure increased initial stability. Generally increasing pressure from 100 to 150 psi increased stability more than increasing pressure from 50 to 100 psi. Each point in Figure 4 represents the average of three stability values that differed from one another by less than 1½ stability units.

The plot of Hveem stability vs initial revolutions shown in the lower half of Figure 4 indicates that increasing initial revolutions from 10 to 20 increased initial stability. The slopes of the lines show that the increase in initial revolutions is most effective in increasing initial stability of specimens compacted at low pressure.

The confinement and deformation characteristics in the gyratory testing machine operating with fixed-roller conditions are different from those encountered in which deformation progresses. Under compaction equipment in the field, the layer of bituminous material becomes more dense. Accompanying this densification is an increase in bearing capacity and lateral support so that subsequent passes with compacting equipment cause successively smaller deformations. Compaction using the fixed-roller operation deforms the specimen by an angular amount at least equal to the gyratory angle, 1^0. Because this movement is greater than that produced by roller or traffic coverages, except perhaps for the first few roller passes, the progression of density and stability in the bituminous specimens is more rapid than that for pavement.

Secondary Compaction. —Figures 5, 6, and 7 may be interpreted as indications of rutting potential due to compaction under varying secondary compaction for mixtures compacted initially for 20 rev. at 1^0 angle using 50, 100 and 150 psi initial pressures. The semilog plots of axial deformation vs number of revolutions record axial deformation as the difference between specimen height when only a static load was applied and specimen height after some number of revolutions. The curves are concave downward only for secondary compaction pressures equal to or greater than the initial compaction pressure, indicating that the rate of axial deformation decreases during secondary compaction if initial compaction pressure exceeds secondary compaction pressure. High tire contact pressures might contribute considerably to densification in cases where initial compaction did not sufficiently densify the mix. In all cases observed in Figures

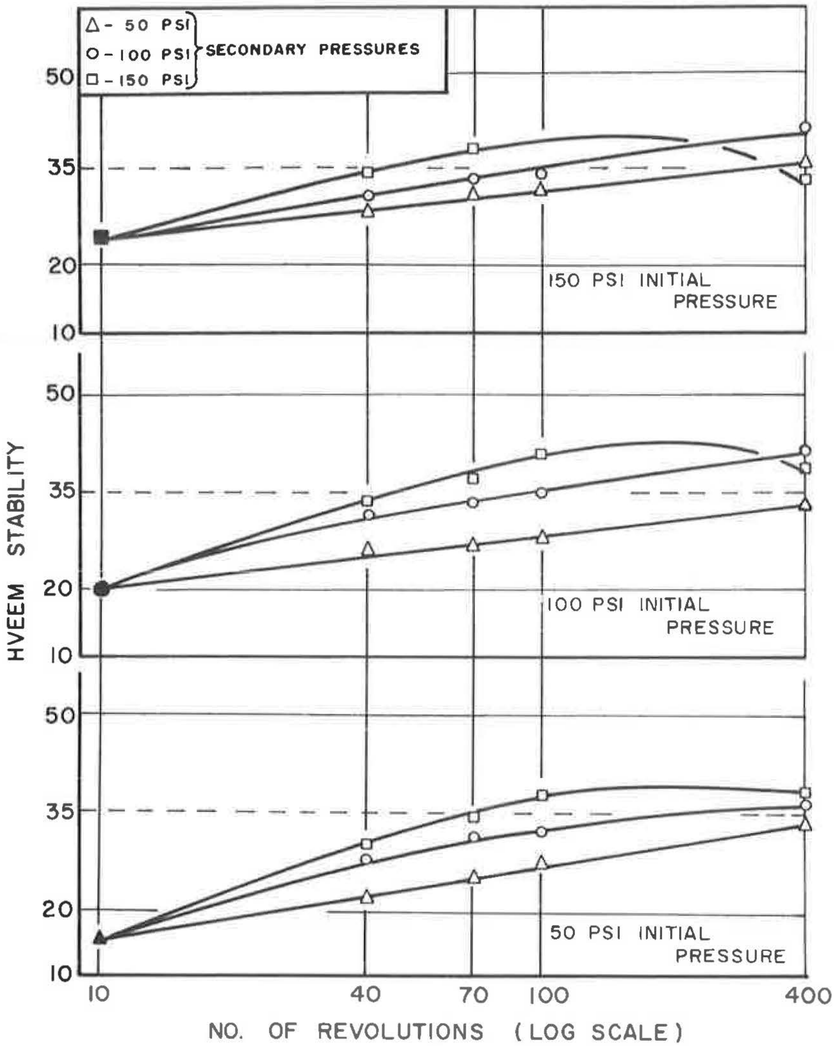


Figure 8. Hveem stability vs no. of revolutions, 10 rev. initial compaction, Fuller's maximum density gradation, 4 percent asphalt.

5 to 7, axial deformation increased, indicating that specimen confinement in the compaction mold was sufficient to prevent particle orientation that would have resulted in a decrease in unit weight.

The number of secondary revolutions was varied to simulate traffic coverages and to obtain an estimate of the variation of specimen stability with time under traffic. Figures 8, 9, 10, and 11 present semilog plots of Hveem stability vs number of revolutions for all 192 specimens compacted using fixed-roller operation. The solid black symbols in each figure represent the values of initial stability determined experimentally from the average of three stability measurements for each initial pressure—50, 100, and 150 psi. Other symbols represent only a single stability determination; however, duplicate determinations were made for those cases where a stability decrease occurred with additional revolutions. The graphs approximate straight lines for low pressures up to approximately 100 rev. and, thus, the relationship would be parabolic on an arithmetic plot.

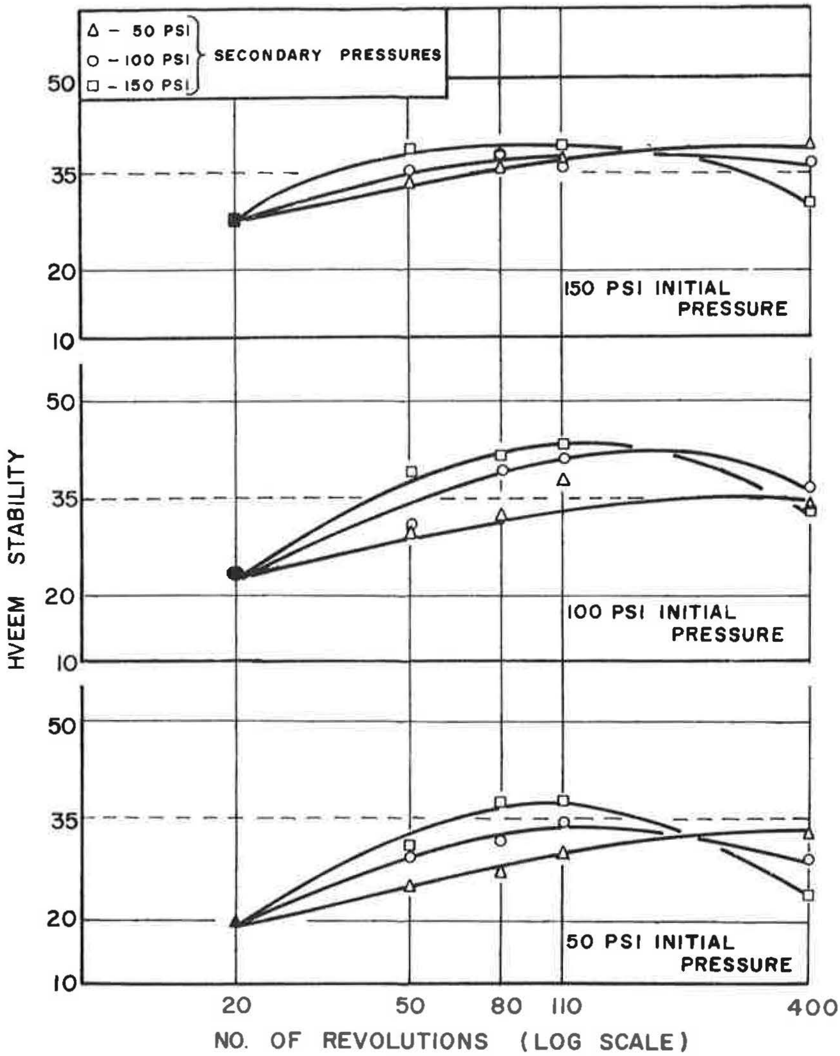


Figure 9. Hveem stability vs no. of revolutions, 20 rev. initial compaction, Fuller's maximum density gradation, 4 percent asphalt.

Gradation. —For gradation D (Figs. 10, 11) increasing revolutions increased stability, but for the Fuller gradation (Figs. 8, 9) stabilities for the high secondary pressures decreased with increasing secondary revolutions. This measurement may provide a relative index of resistance to loss of stability under traffic. For the Fuller gradation decreases in stability may be noted at the 400 rev. level for both 10 and 20 rev. initial compaction. In general, greater decreases in stability occurred in specimens compacted using higher secondary compaction pressures. This result should be expected from energy considerations, i. e., because compaction is an energy-consuming process the results of compaction should be measureable in energy units. Most specimens compacted initially for 20 rev. had somewhat lower stability values after 400 rev. than specimens compacted initially for 10 rev. Because the only difference between the 10 and 20 rev. initial compaction was the amount of compaction that occurred at the initial compaction temperature (225 F), it was concluded that the difference in compaction temperature was responsible for the apparent differences in stability and in resistance to loss in stability. No detailed attempt was made in this study to analyze the effects

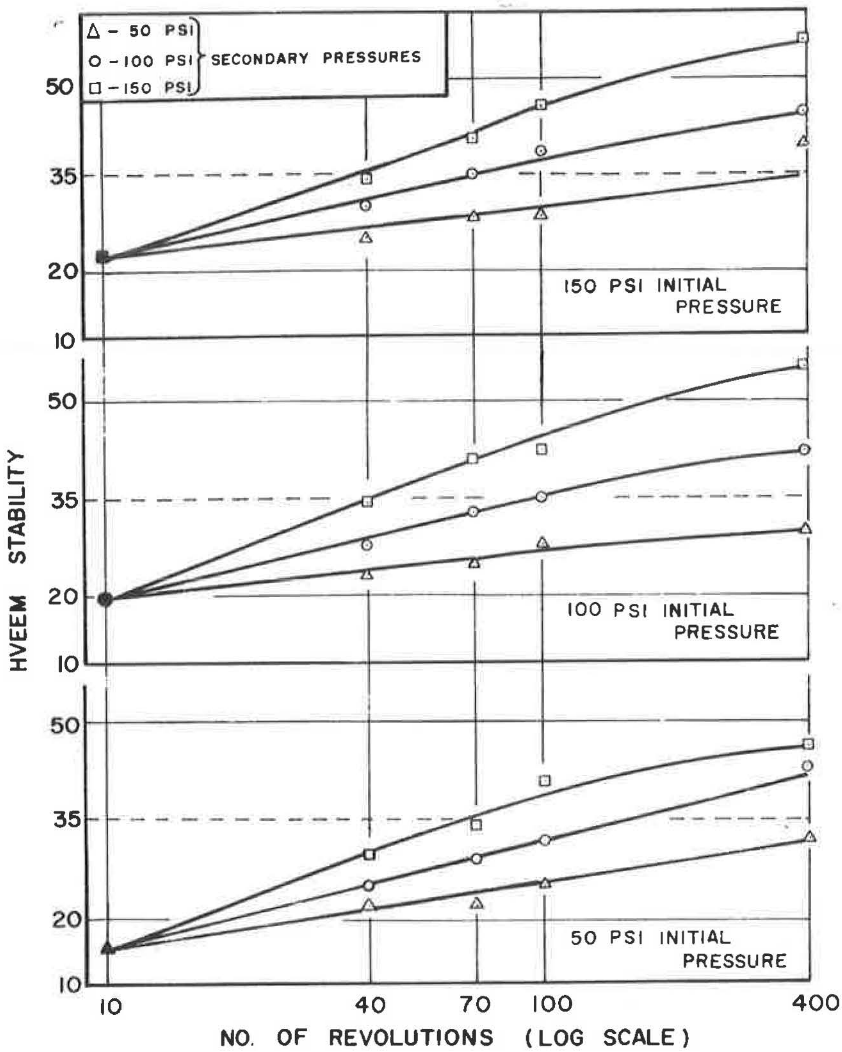


Figure 10. Hveem stability vs no. of revolutions, 10 rev. initial compaction, gradation D, 4 percent asphalt.

of compaction temperature on specimen stability; however, the stability difference observed indicates that some type of compaction temperature specification is necessary to insure uniformity of compaction. Asphalt in thin films exhibits a greater resistance to compaction at low than at high temperatures. For the Fuller gradation, increased initial compaction decreased the secondary compaction that could be applied before loss in stability occurred.

Design Procedures

Additional tests were performed to compare selected laboratory design test characteristics for gyratory- and kneading-compacted specimens.

Design of Dense-Graded Mixes.—Figure 12 presents a semilog plot of percent voids vs secondary number of revolutions for 20 rev. initial compaction of the Fuller gradation mixture with 4 percent asphalt. Comparison of Figure 12 with Figure 9 shows that when degree of compaction of this mixture is such that the void content is less than 2 percent, additional compaction will result in a decrease in stability. A good corre-

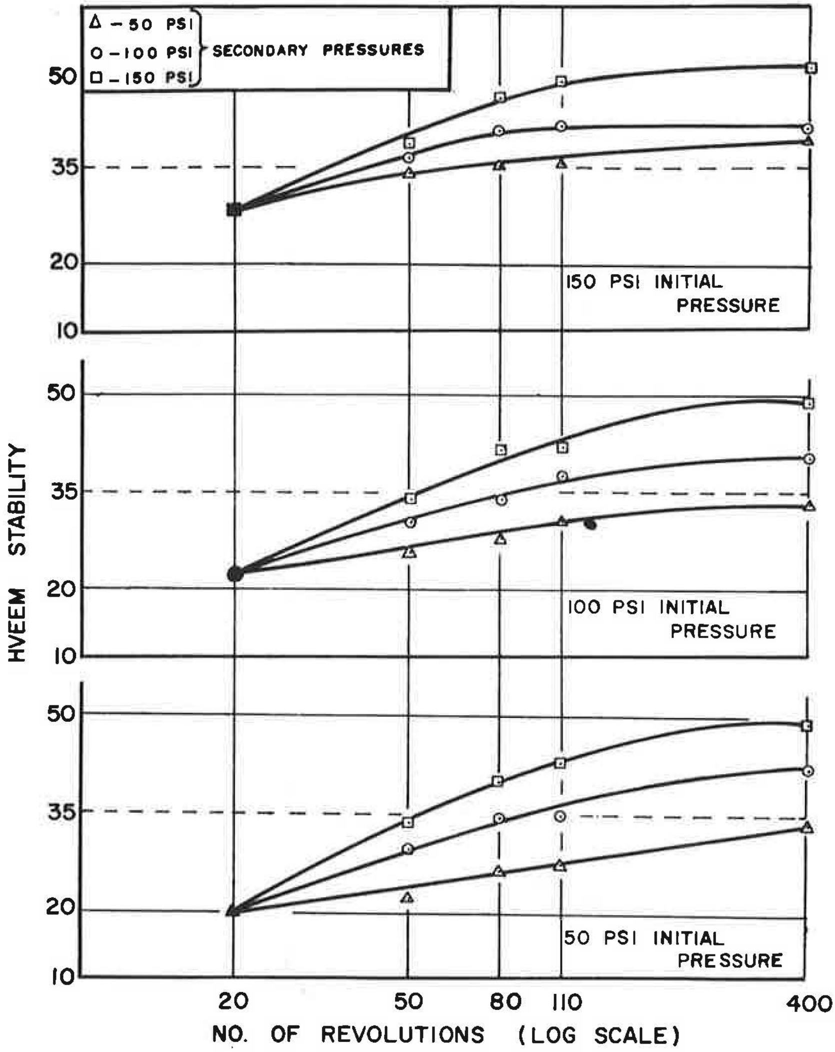


Figure 11. Hveem stability vs no. of revolutions, 20 rev. initial compaction, gradation D, 4 percent asphalt.

spondence between decrease of void value to less than 2 percent and widening of the gyrographs was also indicated. Some typical gyrographs are presented in Figure 13.

To relate the gyratory design technique to a standard design procedure, six specimens of the Fuller gradation were prepared by the standard kneading compaction technique specified in the Hveem design procedure. A plot of Hveem stability and percent voids vs percent asphalt (Fig. 14) indicates that 4 percent is the maximum asphalt content that this mixture can accommodate and remain stable under the compactive effort applied. Figures 8 and 9 reinforce this conclusion. The rather steep slope of the stability vs asphalt content curve (Fig. 14) indicates that the mix is quite sensitive with respect to amount of asphalt and infers mixture sensitivity with increased compaction. From this it was concluded that a design procedure utilizing the widening gyrograph concept was adequate for specifying asphalt content of dense-graded mixes.

Design of Open-Graded Mixes. --To study the possibility of using the gyratory testing machine to select an optimum asphalt content for open-graded mixes, 60 gradation D specimens varying in asphalt content from 4 to 7 percent were compacted in the

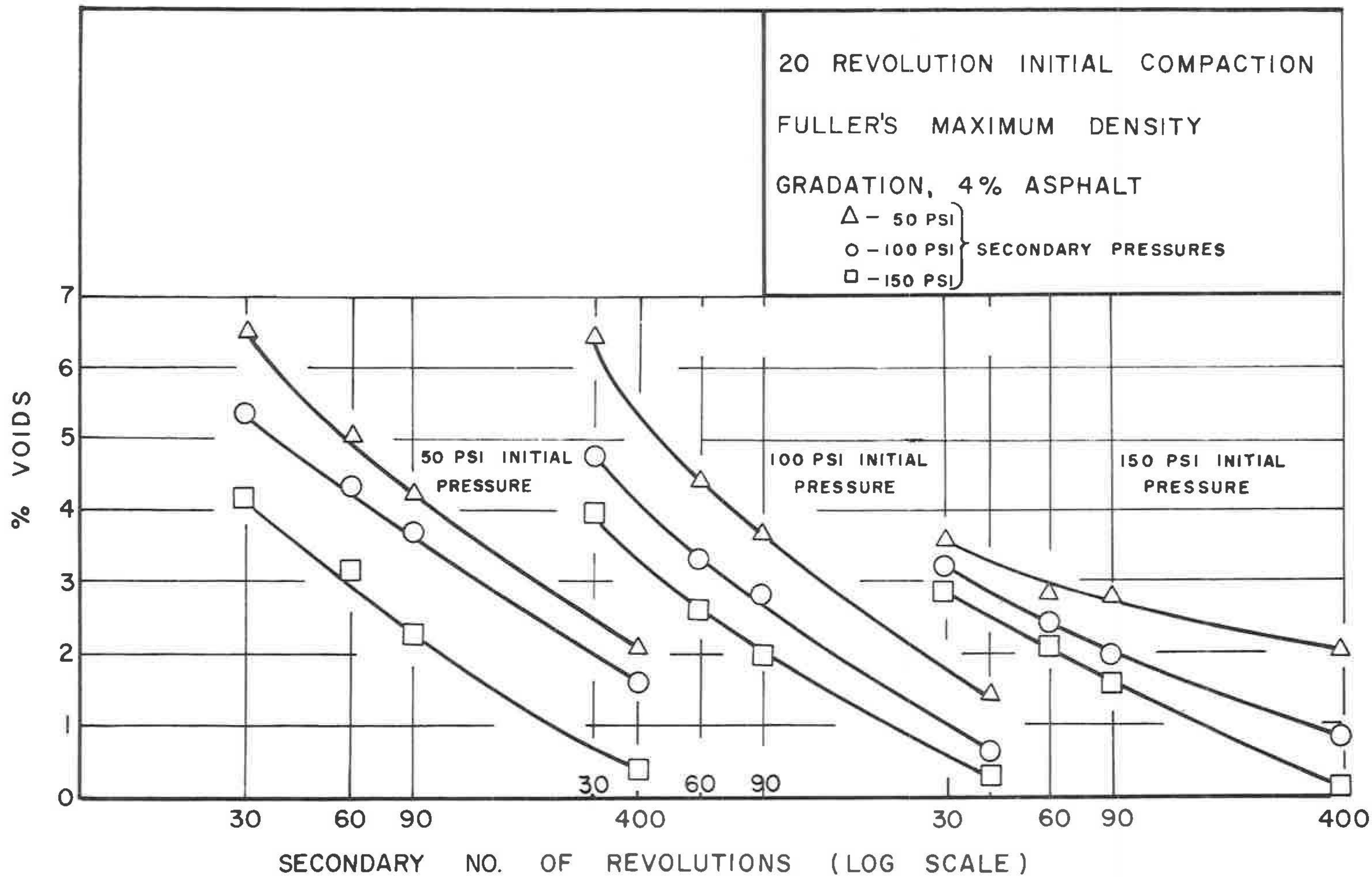


Figure 12. Percent voids vs no. of revolutions.

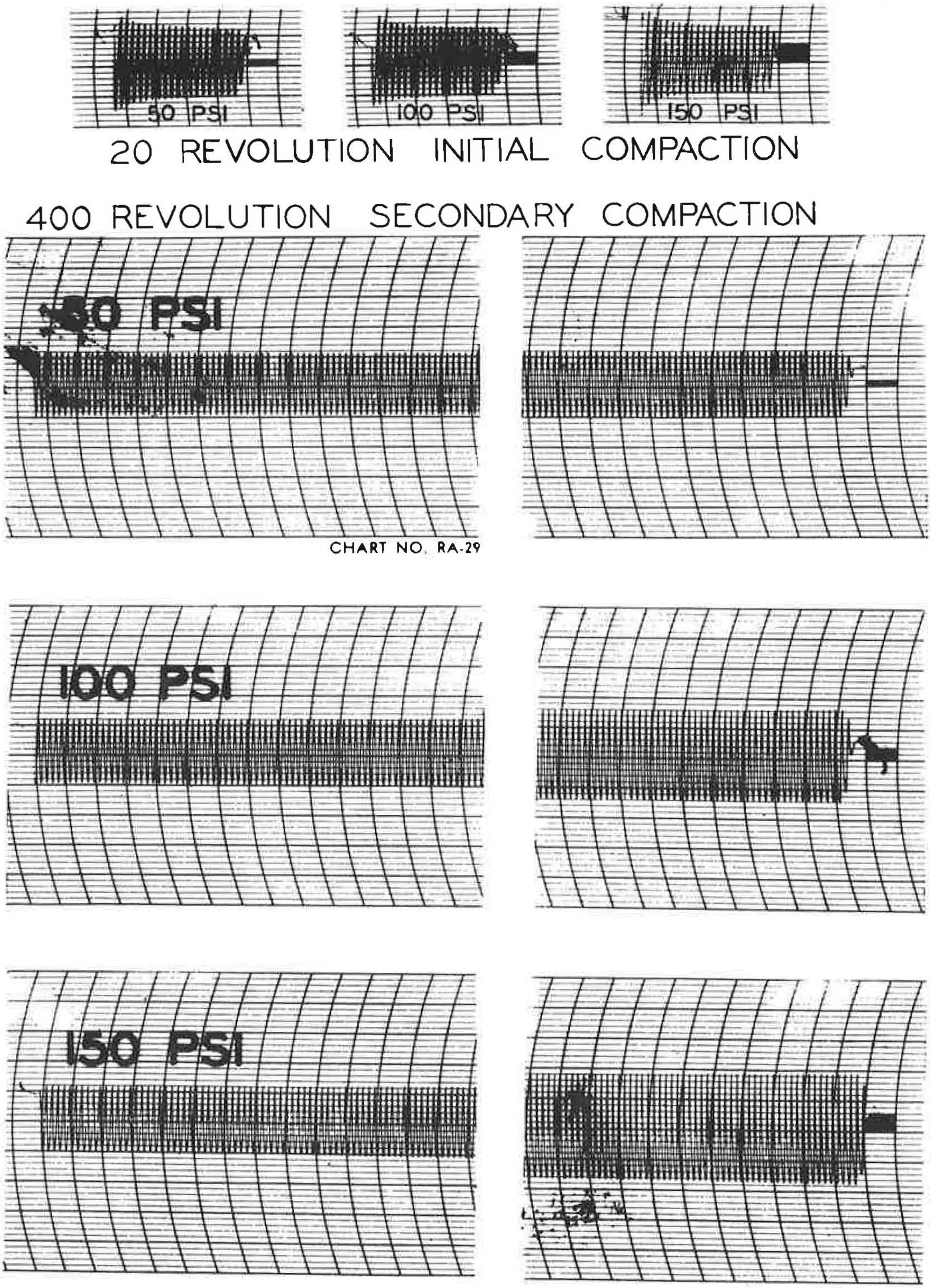


Figure 13. Typical gyrographs--fixed roller operation, Fuller's maximum density gradation.

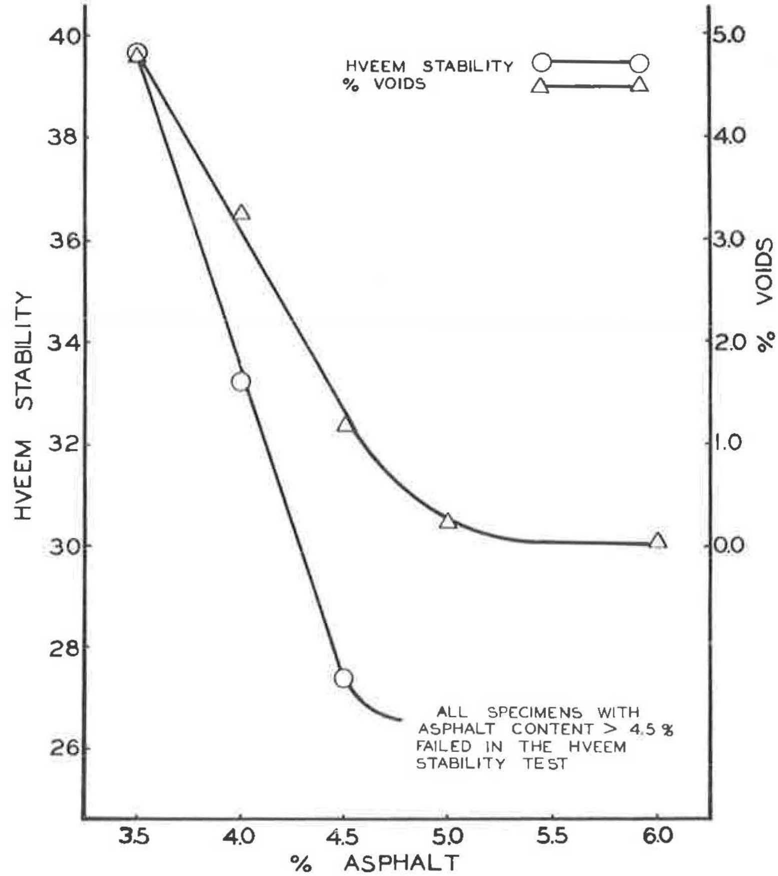


Figure 14. Hveem stability and percent voids vs percent asphalt, Fuller's maximum density gradation, kneading compaction.

gyratory testing machine and tested in the stabilometer. Figure 15 shows semilog plots of Hveem stability vs number of revolutions for these specimens. In each case stability values increased with increased number of revolutions; however, stability at 400 rev. decreased with increasing asphalt content for 150 psi secondary pressures. For all 60 specimens there was no widening of the gyrographs with increasing number of revolutions.

These results were also compared with those from four specimens compacted by the standard Hveem procedure. Figure 16 is a plot of Hveem stability and percent voids vs percent asphalt which can be compared to Figure 14. Stability values for kneading-compacted specimens shown in Figure 16 are much lower than the maximum stability values shown in Figure 15 for gyratory-compacted mixtures of the same composition. No indication of a critical asphalt content was evident from either stabilometer values or widening gyrographs for specimens of gradation D compacted by gyratory compaction up to 400 rev.

Figure 16 shows that an asphalt content of 5.5 percent is required for gradation D to obtain the 4 percent voids generally desired in the Hveem design procedure. However, 5.5 percent asphalt would yield a stability of only 24, less than that required for light traffic by Hveem criteria. Values of percent voids in Table 9 for gradation D gyratory-compacted specimens of 6 percent asphalt were 3.7, 3.2, and 2.4 percent, respectively, for secondary pressures of 50, 100, and 150 psi and 400 rev. Figure 15 shows that this compactive effort yields stabilities of 35 to 45. For this same range of percent voids, Figure 16 shows that kneading compaction yields stabilities of less than 30.

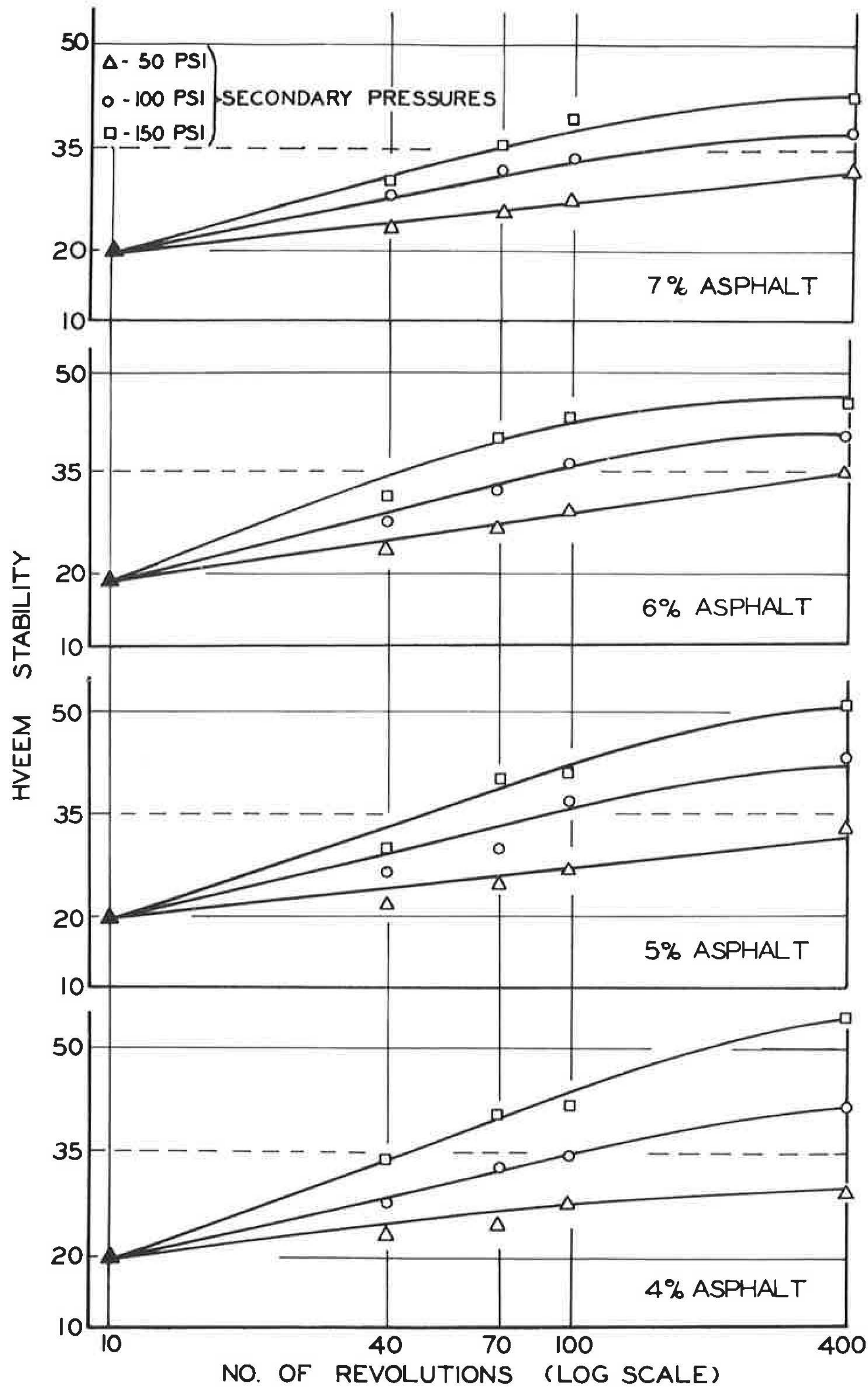


Figure 15. Hveem stability vs no. of revolutions, 10 rev., 100 psi initial compaction, gradation D, varying asphalt content.

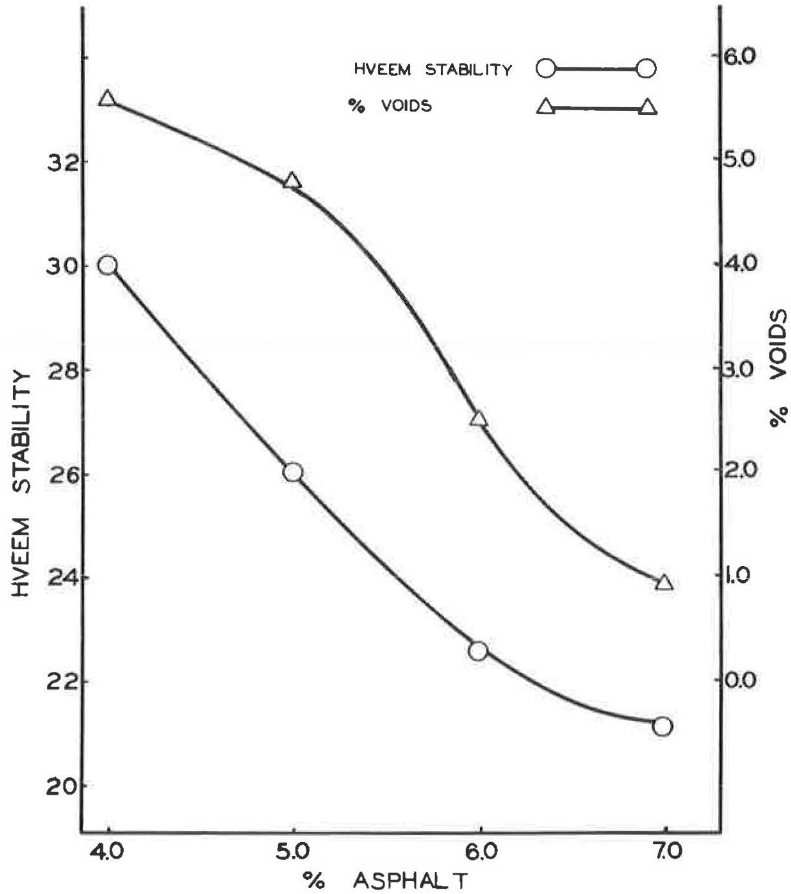


Figure 16. Hveem stability and percent voids vs percent asphalt, gradation D, kneading compaction.

Variation of Unit Weight with Specimen Height

By way of possible explanation for the difference in results obtained by kneading and gyratory compaction of the more open gradation, it should be noted that the high kneading foot pressures specified by the Hveem design kneading compaction procedure produced considerable degradation in specimen tops. Although the percent voids in specimens compacted by the two machines might be equal, this could represent an average of a low-void mix in the top and a high-void mix in the bottom of the kneading-compacted specimens.

To support this theory, statistical analyses were made of variation in unit weight with specimen height for kneading- and gyratory-compacted specimens. Eighteen gradation D specimens containing 4 percent asphalt were compacted in the gyratory testing machine using 10 rev. of fixed-roller operation at 100 psi and a 1^{0} angle of gyration. The 18 specimens were then divided into two groups of nine specimens each. The first group received secondary

TABLE 9

PERCENT VOIDS FOR GRADATION D[a]

Secondary Pressure (psi)	Secondary Revolutions			
	30	60	90	400
50	8.1	7.4	6.5	3.7
100	6.9	5.7	4.9	3.2
150	5.7	4.9	4.1	2.4

[a] Six percent asphalt, 10 rev., 100 psi initial compaction.

TABLE 10

UNIT WEIGHT GRADIENT
OF GYRATORY-COMPACTED SPECIMENS[a]

Secondary Compaction at 1°		Bulk Unit Weight (pcf)		
Psi	Revolutions	Top	Bottom	Top-Bottom
50	60	137.3	136.0	1.3
100	30	137.9	137.3	0.6
100	60	138.5	138.5	0.0
100	90	138.5	137.9	0.6
150	30	136.0	138.5	-2.5
150	60	141.0	140.4	0.6
150	90	140.4	141.0	-0.6
—	—	134.2	132.3	1.9
—	—	134.2	135.4	-1.2

[a]Gradation D, 10 rev., 100 psi, 1° initial compaction, 4 percent asphalt.

TABLE 11

UNIT WEIGHT GRADIENT
OF GYRATORY-COMPACTED SPECIMENS[a]

Pressure (psi)		Bulk Unit Weight (pcf)		
Initial	Secondary[b]	Top	Bottom	Top-Bottom
50	50	141.4	143.1	-1.7
50	100	145.3	147.1	-1.8
50	150	147.0	149.8	-1.8
100	50	144.6	148.0	-3.4
100	100	146.6	148.8	-2.2
100	150	148.2	149.8	-1.6
150	50	145.0	146.0	-1.0
150	100	146.1	147.3	-1.2
150	150	149.6	148.8	0.8

[a]Gradation D, 10 rev., 1° initial compaction, 4 percent asphalt.
[b]At 1° and 390 rev.

compaction up to a maximum of 90 rev. Specific compaction conditions for each specimen are given in Table 10. Data showed that differences in bulk unit weights between specimen tops and bottoms were insignificant.

The second group of nine specimens received 390 secondary revolutions under conditions of fixed-roller operation at the pressures given in Table 11. For this group of specimens, specimen bottoms had higher unit weights than specimen tops. The average difference was 1.56 pcf. Gradation D was used for this test because the more open mixture might reflect more markedly the existence of a unit weight gradient.

Unit weight gradient of seven specimens compacted in the gyratory testing machine using the air-filled upper roller was also studied. For this type of operation, specimen bottoms were, on the average, 1.1 pcf heavier than specimen tops (Table 12).

TABLE 12

UNIT WEIGHT GRADIENT OF GYRATORY-COMPACTED SPECIMENS[a]

Secondary Compaction Pressure[b] (psi)		Bulk Unit Weight (pcf)		
Ram	Air	Top	Bottom	Top-Bottom
50	25	138.5	139.8	-1.3
100	25	138.5	138.5	0.0
150	25	137.3	138.5	-1.3
50	50	140.4	141.6	-1.2
100	50	143.5	144.1	-0.6
50	12	135.4	137.9	-2.5
100	12	136.0	136.7	-0.7

[a]Gradation D, 10 rev., 50 psi, 1° initial compaction, 4 percent asphalt.
[b]Four hundred rev. with air-filled upper roller at variable angle.

For kneading-compacted specimens, unit weight gradient was studied by compacting six gradation D specimens in the kneading compactor using the standard Hveem design compaction procedure. Asphalt contents for these specimens were varied in 0.5 percent increments from 4 to 6.5 percent. For all kneading-compacted specimens the unit weight of specimen tops exceeded the unit weight of specimen bottoms. Results (Table 13) show that the average difference in bulk unit weight was 6.0 pcf and the range was 3.7 to 8.1 pcf. Appearance of the compacted specimens ranged from a powdery, crushed upper surface for the specimen containing 4 percent asphalt to a flushed upper surface for the specimen containing 6.5 percent. No trend relating unit weight gradient and asphalt content was observed.

These results show that type of compaction is important in that it may effect the development of a unit weight gradient; reproduction of unit weight for design purposes must consider the unit weight gradient if an accurate laboratory simulation of the field condition is to be obtained.

Particle Orientation

The effect of gyratory compaction on particle orientation was studied by placing selected pieces of long, slender aggregate, with long axes vertical, in a plastic clay in the gyratory mold. These samples were then compacted for 400, 1,200, and 5,400 rev. using the fixed 1° upper roller in the gyratory testing machine. The gyratory compaction reoriented the aggregate particles into positions where their long axes lie horizontal (Figs. 17, 18), and the aggregate forms concentric circles in this reoriented position. The opportunity for reorientation would be greater in a plastic clay than in an aggregate mix where there is either particle-to-particle contact or particle separation by thin plastic films. Within the limits imposed by these conditions and the confinement of the compaction mold, particle orientation qualitatively similar to that which occurs under traffic appears possible using the gyratory shear method of compaction.

TABLE 13

UNIT WEIGHT GRADIENT OF KNEADING-COMPACTED SPECIMENS[a]

Asphalt Content (%)	Bulk Unit Weight (pcf)		
	Top	Bottom	Top-Bottom
4	146.0	138.5	7.5
4½	147.3	142.9	4.4
5	148.5	142.3	6.2
5½	148.2	142.3	5.9
6	151.0	142.9	8.1
6½	148.5	144.8	3.7

[a]Gradation D.

SUMMARY OF RESULTS AND CONCLUSIONS

The following results and conclusions, derived from the experimental data collected,

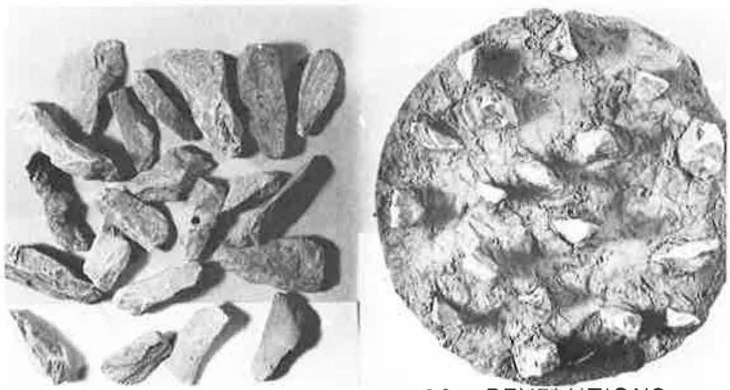


Figure 17. Study of particle orientation.

Figure 18. Study of particle orientation.

are applicable to the materials and testing procedures of this specific research only and may not be extended beyond these limits without appropriate correlation.

1. For the specimens of the Fuller gradation with 4 percent asphalt subjected only to simulated construction compaction in the gyratory testing machine and tested in the Hveem stabilometer, a significant increase in bulk unit weight was effected by the compression imposed on the specimens during testing in the stabilometer. The average increase was 1. 67 pcf.

2. Analysis of variance for the five main factors studied showed all factors were statistically significant in affecting specimen compaction as evaluated by change in stability. Factors in order of importance were: secondary revolutions, initial pressure, secondary pressure, initial revolutions, and gradation. Data from controlled field studies are necessary to determine if a realistic simulation of the pavement condition is effected by this laboratory procedure. However, the same statistical methods can be applied to a field study for an evaluation of field compaction and stability variables.

3. In all cases studied, including both the dense and open gradations at all asphalt contents, increases in initial compaction pressure and number of revolutions increased the initial stability. Increased initial compaction decreased the secondary compaction that could be applied before loss in stability occurred.

4. Axial deformation of specimens under simulated traffic was greater for specimens initially compacted at high pressures. No decrease in unit weight occurred during compaction; confinement in the compaction mold was sufficient to prevent this.

5. Good correlation was obtained between widening of the gyrograph and loss in Hveem stability for the mixture employing the Fuller gradation. Stability values for kneading- and gyratory-compacted specimens compared favorably for the same values of percent voids. Hence, it is indicated that for this laboratory study, good stability and voids correlations were obtained for the dense mix compacted by the kneading compactor and the gyratory testing machine.

6. For gradation D, stability values of kneading-compacted specimens were lower than the stability values of gyratory-compacted specimens when both types had the same percent voids. High stability values were measured for gradation D specimens containing 4 to 7 percent asphalt and compacted to 400 rev. No indication of loss of stability was observed from the widening of the gyrographs. Kneading-compacted specimens of the same open-type gradation had stability values of 30 or less for the 4 to 7 percent range of asphalt content studied. For gyratory- and kneading-compacted specimens, marked differences in stability were attributable to differences in the type of compaction imposed on the specimens. A thorough study of the factors responsible for this discrepancy with respect to the gradation D mixture was not undertaken.

7. Gradation D specimens containing 4 percent asphalt and compacted in the gyratory testing machine had variation in unit weight from top to bottom differing with amount of compaction. Unit weights of specimen bottoms tended to be slightly greater than those of specimen tops.

8. For gradation D specimens with varying asphalt content, kneading compaction as specified in the Hveem design procedure produced specimens whose unit weight increased markedly from bottom to top.

9. Stability values for specimens compacted by the gyratory machine were found to be a function of temperature and mixture composition. Both mixture gradation and asphalt content were factors of composition influencing stability values.

10. Compaction of a plastic clay containing hand-placed pieces of slender aggregate showed that gyratory compaction allowed pieces to orient themselves into horizontal position in a pattern of concentric circles.

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Evaluation of the Cohesiometer Test for Asphaltic Concrete

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The investigation reported herein is a supporting study to a larger project concerned with the modification of the AASHO Road Test findings for use under conditions found in Texas. The objective of this special study was to determine whether cohesiometer test results are significantly related to factors known to affect the performance of asphaltic concrete, to modify the equipment or procedure if necessary, and to evaluate the test for use in the parent project.

The cohesiometer used by and available to the Texas Highway Department was modified slightly and a load-deflection recorder was attached to the unit. The data obtained from the evaluation program have shown that the cohesiometer test results are affected by and are sensitive to mixture variables that exist in asphaltic concrete pavements. An equation defining the cohesiometer's response to a test specimen was derived and verified by test data. Also for use in the parent project a specimen height correction chart was established.

•SINCE THE completion and reporting of the AASHO Road Test, highway engineers have recognized the necessity of translating the findings for local conditions. It is primarily for this reason that project HPS-1-(27) E was initiated in Texas. A special phase of this program is concerned with the determination of the surfacing coefficient for use in the Road Test performance equation and with the cohesiometer test for arriving at a value for this coefficient.

The cohesiometer test was developed by the California Highway Department for use in designing asphaltic mixtures and pavements; however, this test has not been used as a specification requirement for asphaltic surfacings. Several districts of the Texas Highway Department employ a cohesiometer of Texas design for evaluation of pavement materials. This type was selected for use in the study (Fig. 1); however, certain modifications to the standard apparatus were made.

A major modification was the addition of a load-deflection recorder. The recorder is a mechanical one in which a paper tape moves at a rate of 18 in./min and a pen attachment linked to the cohesiometer beam traces a curve on the tape as the beam deflects under load. Other changes from the standard are slight ones, such as (a) the beam is allowed to deflect up to 1½ in., and (b) the variation in the gap distance between the clamp-down plates has been reduced from that of previous models.

Before performing some preliminary testing with the new cohesiometer, several machine characteristics were noted and considered in the testing procedure, as follows:

1. On the specimen deck of the cohesiometer a circle 4 in. in diameter was inscribed for aid in centering the test specimens.
2. Specifications for the cohesiometer required that the gap variability between the specimen clamping plates be restricted to close tolerances.

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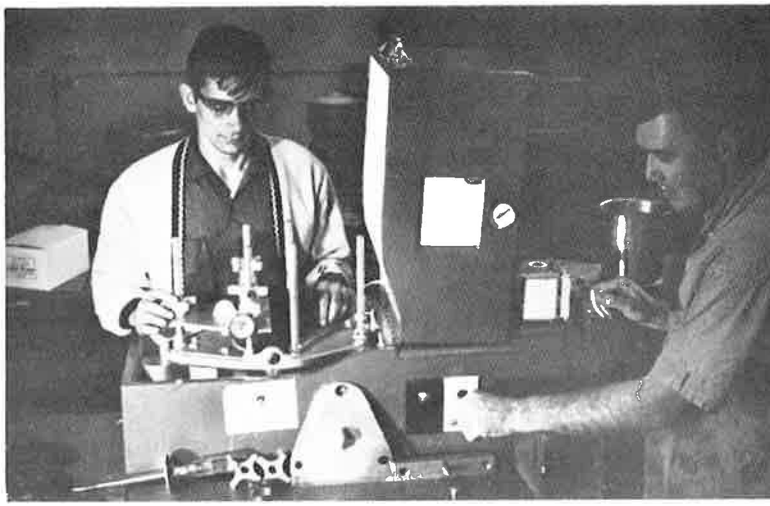

Figure 1. Photograph of cohesiometer.

3. The manufacturer's recommended torque of 25 in.-lb for securing the specimen was found to be excessive for asphaltic concrete specimens. The maximum torque used was 20 in.-lb, and, in some instances, lower torque values were found to be necessary to avoid damaging test specimens.

4. The fixed location of the thermometer for determining cabinet temperature is not considered to be proper. Generally, during use the test temperature is reached sooner at the elevation of the fixed thermometer than at the elevation of the test specimen. For this reason it was necessary to place a thermometer on the fixed-side clamping plate (Fig. 1) for determining and controlling the test temperature.

5. The present design of the cohesiometer utilizes a cam for supporting the loaded end of the cohesiometer beam. The beam deflection recorder responds to the bending of the beam because of its own weight when the end cam support is released. Future designs should eliminate this type of deflection from the load-deflection graph. A typical curve is shown in Figure 2.

6. A sturdier or more rigid construction of the cohesiometer is preferred.

The available literature on this test is limited to procedure and to values obtained for different mixture studies. The common procedure calls for testing a specimen generally 4 in. in diameter, 2 to 2½ in. high, at a temperature of 140 F, and using a rate of loading of 1,800 gm/min. The loading is stopped when the end of the beam deflects ½ in. The load corresponding to the ½-in. deflection is corrected for specimen height to obtain the cohesiometer value.

The asphaltic mixture characteristic evaluated by the cohesiometer may be related to flexural strength of the material and, therefore, information on this property is required for mixture design and evaluation. Use of the cohesiometer under existing procedure has shown that for normal asphaltic concrete there is no visual evidence of failure of the specimen and that the available height correction factors are not adequate for thin specimens. An objective of this study was to determine a method for transforming the test value of specimens of different heights to that value of a specified height.

Because of the lack of information on the theory originally hypothesized for the response of the cohesiometer to test conditions, an hypothesis was stated for the present study and a model equation for cohesiometer response was obtained for verification. The hypothesis is based on the flexural nature of the test. Test conditions are shown in Figure 3. The external moment due to W is given by

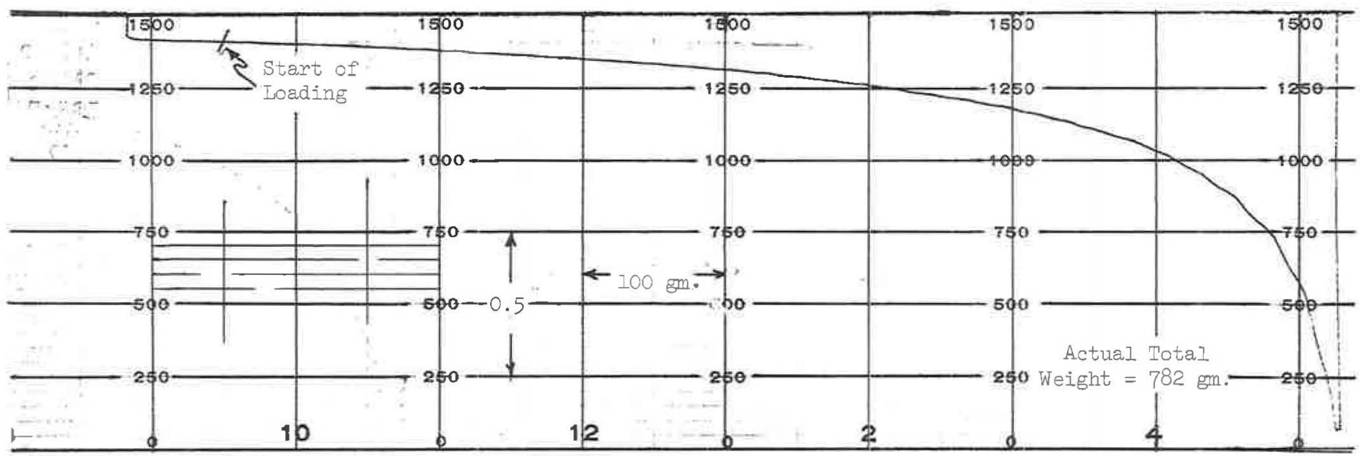

Figure 2. Typical load-deflection curve from cohesiometer test.

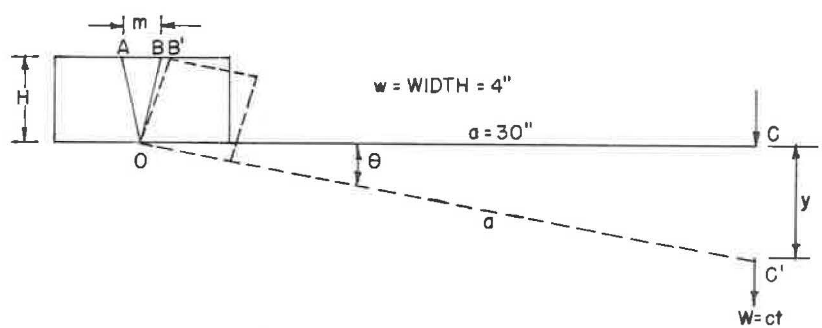

Figure 3. Conditions of cohesiometer test.

$$M_W = a c t \cos \theta \tag{1}$$

in which c is the rate of loading in gm/sec and t is the period of loading in sec. The resistance to the external moment comes from the material in the wedge OAB and is taken to be some function of the original wedge dimensions, H, m, w, of some parameter, K, representing all properties of the material, and also of the instantaneous angular velocity, $d\theta/dt$ at which the deformation occurs in order to account for rate of loading effects. This resistance is represented by

$$M_r = f(H, m, w, K, d\theta/dt) \tag{2}$$

As a first approximation it is assumed that the resisting moment is directly proportional to the instantaneous angular velocity and that dimensions m and w (w = 4 in.) will not be varied for the present; also the length a (a = 30 in.) will be a constant. Under these conditions Eq. 2 becomes

$$M_r = f(H, K)\, d\theta/dt \tag{2a}$$

Neglecting the momentum of moving parts, the external and resisting moments are equated in

$$M_W = M_r; \; a c t \cos \theta = f(H, K)\, d\theta/dt \tag{3a}$$

Separation of variables yields

$$\frac{d\theta}{\cos \theta} = \frac{a c t\, dt}{f(H, K)} \tag{3b}$$

With integration Eq. 3b becomes

$$\log_e \frac{(1 + \sin \theta)}{(1 - \sin \theta)} = \frac{a c t^2}{f(H, K)} \tag{4}$$

Substitution of y/a for $\sin \theta$, and W/c for t results in

$$\log_e \frac{(a + y)}{(a - y)} = \frac{a W^2}{c f(H, K)} \tag{5}$$

which can then be written

$$\log \frac{(a + y)}{(a - y)} = AW^2; \; A = \frac{n a}{c f(H, K)} \tag{6}$$

by changing the base of the logarithm and collecting terms. Eq. 6 indicates that its graph on coordinates of $\log [(a + y)/(a - y)]$ vs W^2 should be a straight line of slope A. Preliminary test data have shown the inital portion of this graph to be a straight line (Fig. 4) and, therefore, there is agreement between the test data and the model equation presented. In Figure 4 the deflection value y is used instead of $\log [(a + y)/(a - y)]$ to simplify the plotting operation and yet still show the general shape of the graph for Eq. 6. For the values of y considered, a practical linear relationship exists between log [(30 + y)/(30 - y)] and y. Failure of the model and the specimen is assumed to occur when the plotted data cease to lie on the initial straight line of the graph. It is interesting to note that the apparent failure of specimens represented in Figure 4 is occurring at a deflection of 0.25 in. as opposed to the original procedure in which 0.50-in. deflection is the failure criterion.

Preliminary testing with the cohesiometer was done on specimens of three different heights and at three different rates of loading. A regression analysis of these data indicated a reliable relationship, $r^2 = 0.915$, between the logarithm of specimen height, H, and that of the product of the slope of the straight line of the proposed model, A, and the rate of loading, c (Table A). This relationship suggests that:

$$cA = F(H, K) = H^b f(K) \tag{7}$$

by assuming $F(H, K) = H^b f(K)$

which in turn yields the relationships

$$\log(cA) = \log f(K) + b \log H \tag{8}$$

and

$$\log(cA) = \log a_0 + b \log H \tag{9}$$

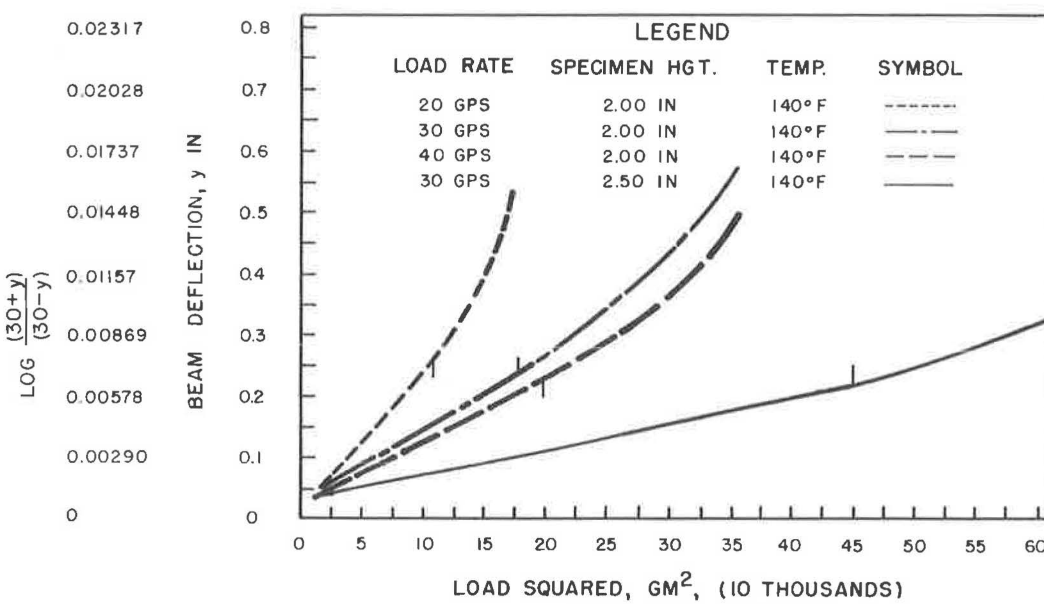

Figure 4. Load-deflection relationship for cohesiometer test.

TABLE 1

GRADATION FOR AGGREGATE BLENDS

Comb. No.	Percent Passing Sieve											
	½ In.	⅜ In.	No. 4	No. 8	No. 10	No. 16	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200
1	100.0	98.4	65.3	50.0	47.5	40.0	25.0	20.0	14.4	11.0	9.5	3.5
2	100.0	98.0	58.0	40.0	38.2	38.0	35.0	28.5	20.0	12.0	10.5	5.0
3	100.0	98.4	65.3	50.0	47.3	38.8	25.0	20.4	17.0	12.0	10.4	3.2
4	100.0	98.0	58.0	40.0	38.6	37.7	35.0	29.0	20.7	12.3	10.9	3.3

Equation 9 is taken as the basic equation for representing a specimen's strength in terms of mixture characteristics, log a_o, and specimen height, H.

This preliminary work was done on specimens made from an actual construction paving mixture in which component variables could not be controlled.

In planning the experiments, it was deemed desirable to investigate the effects of the following factors which are believed to exist and influence the performance of asphaltic concrete surfacings:

1. Aggregate gradation: (a) dense graded, and (b) gap graded;
2. Surface texture of aggregate (- No. 8 sieve size): (a) rough, and (b) smooth;
3. Asphalt content (80 to 100 penetration); and
4. Specimen height and density.

The basic aggregate blends, identified as Combinations 1, 2, 3 or 4 are:

1. Combination 1—dense graded and rough surface texture;
2. Combination 2—gap graded and rough surface texture;
3. Combination 3—dense graded and smooth surface texture; and
4. Combination 4—gap graded and smooth surface texture.

The rough-textured aggregate blends were obtained by combining a rounded gravel and hard limestone screening. The limestone screening furnished most of the minus No. 8 size particles.

The smooth-textured aggregate blends were made by blending the same gravel as in the previous aggregate mixture, concrete sand and field sand. To facilitate the duplication of gradations for both rough- and smooth-textured aggregates, it was thought that the use of the gravel for the plus No. 8 size for all combinations would not greatly minimize the surface texture effect on test values. Table 1 gives the gradations of the various blends obtained by computation for blending the different aggregates, and Figure 5 shows graphically the size distribution obtained after actual blending for the dense- and gap-graded combinations of rough-textured aggregates.

Evaluation of the different asphalt aggregate combinations was made according to the Texas Highway Department method in which specimens are formed by gyratory shear compaction. The results of these tests are presented in Table 2. The cohesiom-

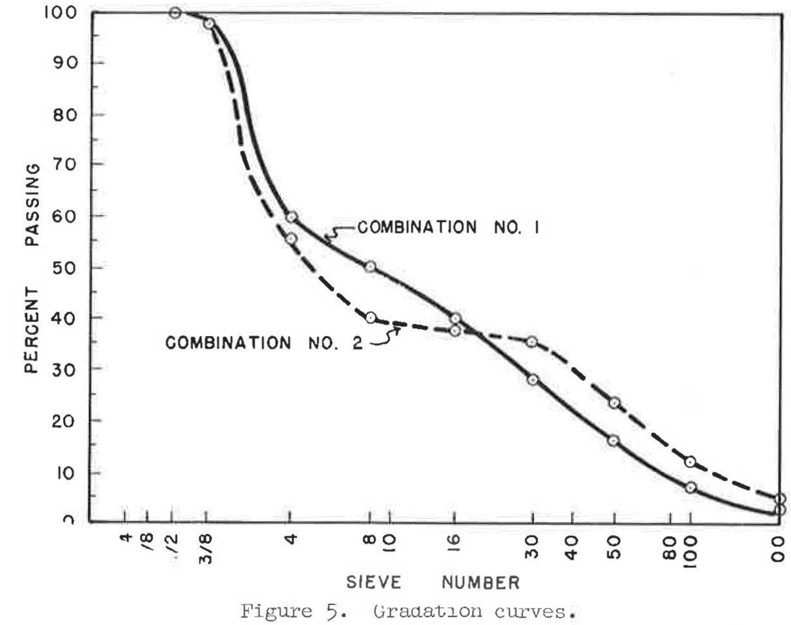

Figure 5. Gradation curves.

eter values were obtained by the original method of testing. All asphaltic mixtures contained an 85 to 100 penetration grade asphalt which met the State's specifications.

EXPERIMENTAL WORK

Three experiments were set up for studying the response of the new cohesiometer to the different variables considered.

Experiment 1 involved primarily the effects of temperature on the strength of test specimens. The variables in this testing program were:

1. Temperature—74, 90, 105, 120 and 140 F;
2. Asphalt content—5.0 and 5.5 percent; and
3. Specimen height—1.5 and 2.0 in.

For these specimens, aggregate Combination 1 was used, and the rate of loading with the cohesiometer was the standard 30 gm/sec.

Experiment 2 included the variables:

1. Aggregate—4 combinations;
2. Asphalt content—4.5, 5.0 and 5.5 percent; and
3. Compactive effort—3 levels.

In view of background experience with the cohesiometer test and study of the first experiment, testing conditions were standardized to a test temperature of 140 F and a loading rate of 30 gm/sec.

The objective of Experiment 3 was to determine a relationship between a value representing the strength of a specimen and the height of the test sample. The establishment of this relationship is necessary to allow comparison of cohesiometer values for different mixtures. Because one curve of these variables of strength and height would not satisfy the needs, the following variables were incorporated:

1. Specimen height—1.50, 1.75, 2.00 and 2.25 in.;
2. Compactive effort—2 levels;
3. Asphalt content—2 levels; and
4. Aggregate—Combinations 1, 2, 3 and 4.

In effect, these variables represented 16 different pavement mixtures with differences other than thickness.

TABLE 2

DESIGN CHARACTERISTICS OF ASPHALTIC CONCRETE MIXTURES WITH OA-90 ASPHALT

Combination No.	Asphalt Content (%)	Air Content (%)	Hveem Stability	Cohesiometer Value (gm/in. width/3-in. height)
1	4.5	5.6	61	212
	5.0	3.4	51	293
	5.5	2.5	40	303
2	4.5	6.4	57	177
	5.0	4.3	51	249
	5.5	3.5	33	254
3	4.25	4.2	44	127
	4.5	3.8	43	150
	4.75	3.3	35	114
4	4.6	4.7	36	103
	4.85	3.5	38	133
	5.10	2.9	38	157

DISCUSSION OF RESULTS

In the basic strength equation for the cohesiometer test (Eq. 9), cA can be simplified by the elimination of c if a standard rate of loading is specified for the test. Also, Figure 4 indicates that the load at the end of the straight-line portion (representing failure of a specimen) might be correlated with the slope of that line and this suggests the direct use of the failure load instead of the slope of the line in Eq. 9. Figure 6 shows that a correlation between failure load, W_f, and slope, A, does exist; that is, that specimens having different failure loads are not likely to have the same value for slope A. The constants of the equation $W_f \cong (8000)/(A^{0.600})$ were determined directly from the graph and do not represent "best-fit" values. The term c is kept in Eq. 9 for flexibility, should variations in loading rate be desired in future work. For the present, the value of slope A, instead of failure load W_f, will be used because the evaluation of A is felt to be more exact than establishing the location of the end of the straight-line portion of curves such as shown in Figure 4. However, it is possible that testing variations may be larger than differences obtained in the use of A or W_f in Eq. 9.

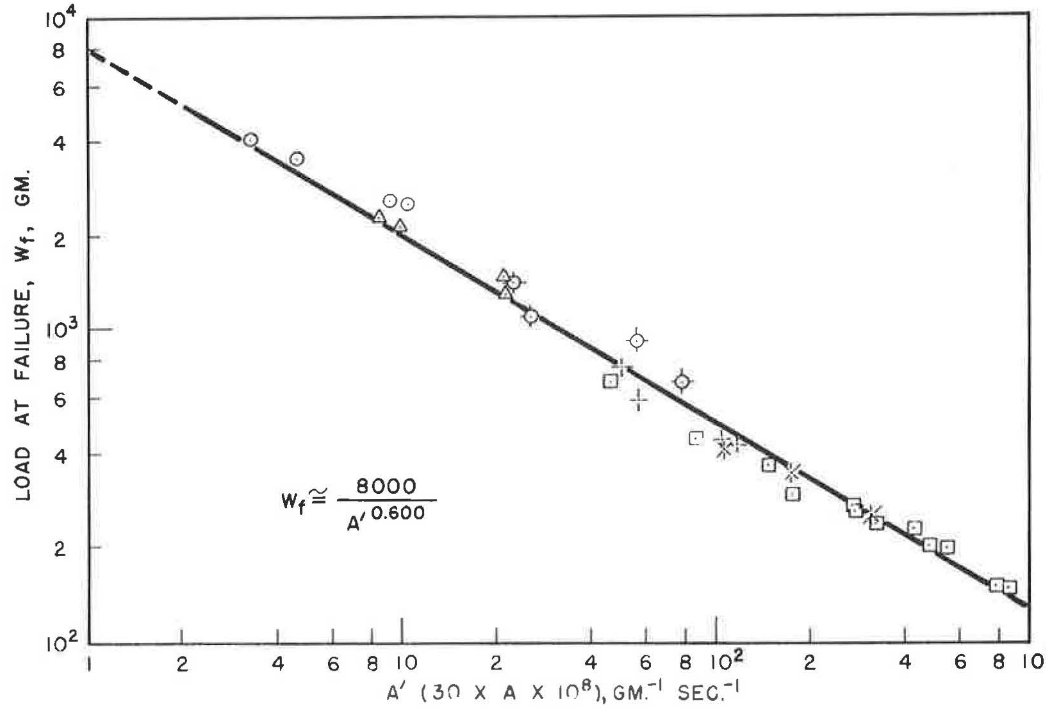

Figure 6. Relationship between load at failure, W_f and A'.

The results obtained in the preliminary testing with the cohesiometer are presented in Table A. These data have indicated the relationship

$$\log (cA \times 10^8) = 3.349 - 4.473 \log H \tag{10}$$

in which

A = slope of the initial straight-line portion of a log [(30 + y)/(30 - y)] vs W^2 plot;
y = deflection in in. at the end of the cohesiometer beam corresponding to the load W in gm;
c = rate of loading in gm/sec; and
H = height of specimen in in.

As shown, the correlation coefficient, r^2, had a value of 0.915.

The basic data for Experiment 1 are given in Tables B1 and B2. For ease in tabulating and use, the symbol A' has been substituted for $30 \times A \times 10^8$. The analysis of variance for these data shows that temperature, specimen height, interaction between asphalt content and height, and interaction between asphalt content and temperature had significant effects on the cohesiometer response represented by the value of A'. The lack of significant effect by asphalt content alone can be explained by the fact that in regular testing with the cohesiometer, it has been observed that the strength of specimens increases as asphalt content increases, but only to an optimum amount of asphalt. Increasing the amount of asphalt above such an optimum value results in a decrease of cohesiometer value. Further, for most asphaltic concrete specimens containing asphalt near the optimum amount, the cohesiometer value is not affected to a significant extent by slight variations of asphalt content. This behavior is illustrated by the cohesiometer values presented in Table 2.

As mentioned in discussing Experiment 1, the effect of temperature was significant;

however, a limited study of the comparison between A′ and temperature did not show a distinct discontinuity near the softening point temperature of 115 F for the asphalt used. Also a study of the standard deviations for the different A′ values did not indicate differences within these values that could be attributed to test temperatures. Perhaps a more meaningful way of expressing the variations of values is by the coefficient of variation which is the standard deviation divided by the mean value and is usually expressed in terms of percent. The coefficients of variation for A′ of specimens 2.00 in. high and containing 5.0 and 5.5 percent asphalt averaged 11.7 and 8.5, respectively, for the test temperature range from 74 to 140 F; these values for the temperature of 140 F were 11.4 and 8.7. For these reasons and because of experience in this area of testing, a temperature of 140 F was chosen for a standard test.

In Experiment 2 the variables considered were aggregate, asphalt content, and compactive effort.

The standard Texas Highway Department method of asphaltic concrete laboratory compaction requires that gyratory shear be imparted to the mixture until a particular strength of mixture, or "end point," is obtained. The end point is reached when one stroke of the standard jack handle raises the ram pressure to 100 psi. To achieve a variation in density for different compacted mixtures, the molding procedure was modified by setting 50 and 200 psi as end points.

Table C gives the values of A′ obtained in this program. It can be seen that the range of compactive effort used caused significant changes in strength as indicated by A′ for all mixtures. A review of Figures 4 and 6 shows that a high value of A′ is associated with a weak specimen. An increase in compactive effort may either increase or decrease the value of A′ depending on the amount of asphalt and the aggregate combination contained in the specimen.

The use of compactive effort for showing these effects may be questioned by those who would prefer to make the comparison on the basis of void content or asphalt film thickness; however, the density variations were not made to establish a design criterion but to study the cohesiometer's response to changes in density.

The data also show that the aggregate combinations employed affected the results of the test. The dense-graded mixtures were generally stronger than the gap-graded ones but were more susceptive to decrease in strength at the higher asphalt content with an increase of compactive effort. Rough-textured aggregate produced specimens stronger than those containing smooth-textured aggregate. However, a combination of gradation and texture can be found such that a well-graded smooth-textured aggregate mixture (Combination 3—4.5 percent asphalt, 200 psi end point) can be stronger than a gap-graded rough-textured aggregate mixture (Combination 2—4.5 percent asphalt, 50 psi end point).

The ultimate desire of the research was to establish a means by which different asphaltic concrete surfacing materials can be compared in terms of some characteristic parameter. Inasmuch as asphaltic surfacings are of different thicknesses, the method established should evaluate the mixture's characteristic parameter in relation to thickness. And further, because all asphaltic pavements are not made from the same materials, this same method of evaluation should be responsive to differences in mixtures. These considerations were the basis for choosing the variables of Experiment 3. It is recognized that compositional variations of the specimens tested were not as great as those found in actual pavements; however, the strength variations created by changes in thickness, asphalt content, and density are considered to be as great as those found in practice. The results of evaluations from Experiment 3 are given in Table D.

The molding of mixtures by the Texas Highway Department's method resulted in a simple procedure for obtaining specimens of different height but with equal density. An examination of the slopes, b, obtained from a regression analysis of $\log A' = \log a_o + b \log H$ shows a range of values from -2.6 to -4.2. These extreme values of b occur for mixtures that are comparable in strength characteristics and do not represent extremes of strength. A plot of log A′ vs log H for the 16 sets of specimens showed that variations in the slope, b, were not correlated with any of the variables studied, nor with strength. Thus, a constant slope was suggested by the data in this

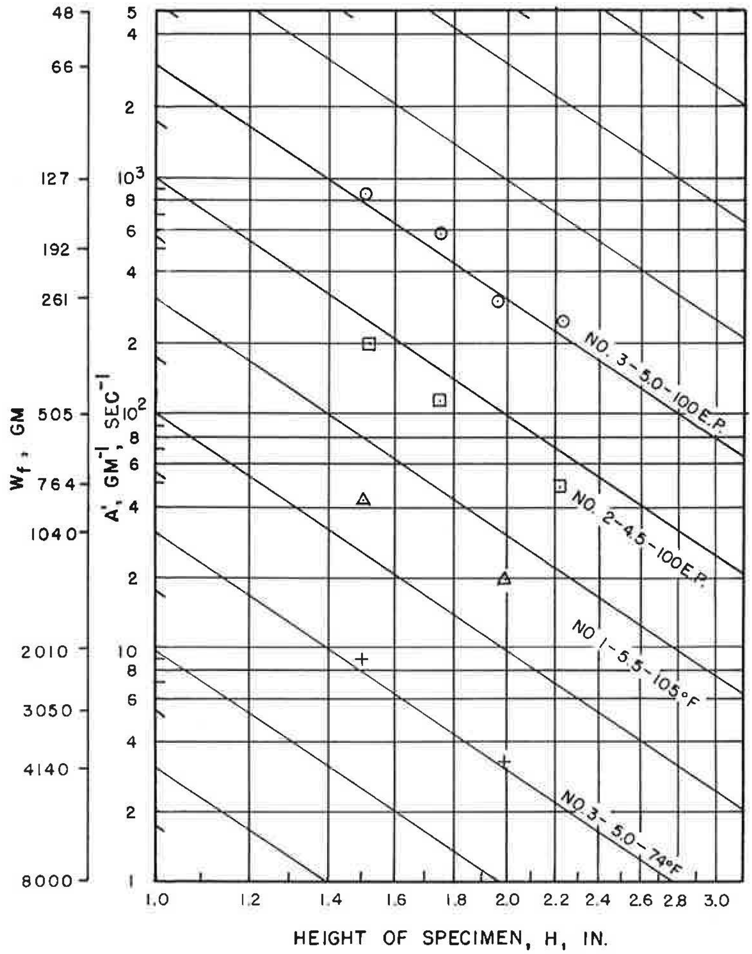

Figure 7. Cohesiometer height correction chart.

experiment and was obtained by averaging the 16 values of b, giving -3.558 with a standard deviation of individual values of 0.698.

The transformation of a cohesiometer test value, A′, for a specimen of a specific height to a standard height specimen can be done by means of Figure 7 which contains logarithmic coordinates of value A′ and H. Also shown is an axis, W_f, in which the value corresponds to the failure load located at the end of the straight-line portion of the y-W^2 plot (Fig. 4). In addition these are data points for different mixtures. Although the data from Experiment 1 were not used to establish the slope of the guide lines, the two bottom sets do appear to follow the trend presented. The use of this chart involves entering into it with values of H and A′ and assuming a standard height of H of 2.00 in. As an example, if a specimen 4 in. in diameter and 1.40 in. high yields a test value of A′ equal to 200, the strength of a standard specimen of such a mixture is determined by locating on the chart a point described by the two given coordinates. From this point a line is followed parallel to the guide lines and intersecting the vertical line representing H = 2.00 in. The ordinate, A′ = 60, of this junction point then indicates the strength as represented by A′ of a standard specimen.

A similar description for height correction can be made for use of the failure load W_f.

SUMMARY AND CONCLUSIONS

The objectives of this study were primarily to evaluate the cohesiometer test for use in project HPS-1(27)E and secondarily to modify the equipment or procedure to achieve these objectives. It has been found that the modified cohesiometer test as described herein yields results that are affected by variables found in asphaltic concrete and which are believed to affect the performance on such pavements. Modifications to the cohesiometer test involved the following items:

1. A load-deflection recorder was attached to the apparatus to obtain a record of beam deflections and corresponding loads during a test.
2. A 4-in. diameter circle was inscribed on the specimen deck to aid in centering a test specimen.
3. The specimen clamping plates were modified to minimize the variability of gap opening.
4. The torque applied to secure a specimen was limited to 20 in.-lb; however, in some instances this value was reduced to as much as 10 in.-lb to prevent damaging a test specimen.
5. The free end of the cohesiometer beam was allowed to deflect 1½ in.

A study of the mechanics of the cohesiometer test led to the derivation of a model equation for defining the cohesiometer's response. Another equation was found to be suitable for establishing a height-correction chart for reducing test values to strengths of specimens of a standard height:

$$\log A' = \log a_o - 3.558 \log H \qquad (11)$$

It is not the intention of this report to set the standard specimen height at 2.00 in. in the evaluation of pavement surfacing to be tested for the parent project, "Application of AASHO Road Test Results to Texas Conditions." It is believed that the standard height for pavement samples should be set in consideration of the average thickness of road samples to be tested and the average height of specimens used in this study.

The cohesiometer test procedure used in this study has been described in detail in a publication of the Texas Highway Department (1).

ACKNOWLEDGMENT

The research presented in this report was done under sponsorship of the Texas Highway Department and the Bureau of Public Roads. Most of the statistical analysis was done under the guidance of Donald E. Cleveland of the Texas Transportation Institute. The basic model equation proposed for the cohesiometer test response was derived by Frank Scrivner of the Texas Transportation Institute.

REFERENCE

1. "Application of AASHO Road Test Results to Texas Conditions." Texas Highway Department Tech. Rep. 3.

Appendix

TABLE A

RESULTS OF PRELIMINARY TESTING WITH COHESIOMETER OF CONSTRUCTION MIXTURE[a]

Specimen Height, H (in.)	Rate of Loading, c (gm/sec)	Slope A × 10^8 (gm^{-2})
1.5	20	24.2
		11.7
		26.6
1.5	30	13.8
		10.5
		8.99
1.5	40	7.64
		9.09
		9.35
2.0	20	5.06
		6.38
		5.88
2.0	30	2.90
		4.56
		3.24
2.0	40	2.78
		1.44
		4.41
2.5	20	1.55
		2.16
		1.19
2.5	30	0.73
		1.54
		1.29
2.5	40	1.13
		1.12

[a]Test temperature, 140 F.

TABLE B1

VALUES OF A′ OBTAINED FROM EXPERIMENT 1[a]

Temp. (F)	5.0% A.C.		5.5% A.C.	
	1.5 In.	2.0 In.	1.5 In.	2.0 In.
74	12.24	2.94	9.63	5.16
	7.95	3.87	11.37	4.26
	7.26	3.00	10.38	4.53
90	20.13	8.64	20.37	9.39
	20.70	8.79	22.35	9.84
	23.82	8.58	20.28	10.56
105	78.00	25.41	48.90	24.87
	69.00	27.54	64.80	20.94
	87.00	24.06	55.50	22.56
120	126.3	49.20	75.30	52.20
	109.8	57.30	109.2	53.40
	110.4	64.20	130.8	44.40
140	223.2	89.70	173.1	104.1
	309.0	105.0	192.0	123.9
	399.6	112.8	158.1	116.1

[a]Rate of loading of 30 gm/sec, A′ = 30 × A × 10^8 / gm-sec.

TABLE B2

ANALYSIS OF VARIANCE FOR CODED DATA[a] EXPERIMENT 1[b]

Source of Variation	D. F.	Sum of Squares	Mean Square	Variance Ratio
Percent asphalt, P	1	7,194.15	7,194.15	1.82
Height of specimen, H	1	2,008,242.15	2,008,242.15	507.91[c]
Temperature, T	4	15,265,164.77	3,816,291.19	965.18[c]
P × H	1	42,400.42	42,400.42	10.72[d]
P × T	4	96,064.10	24,016.03	6.07[c]
H × T	4	31,449.10	7,862.28	1.99
P × H × T	4	36,022.49	9,005.62	2.28
Error	39	154,204.67	3,953.97	
Lost observation	1			
Total	59	17,640,696.85		

[a]Coded data = (log 30 × A × 10^8) 1,000 - 400.
[b]Est. variance = 3953.97 = 0.00395397; σ = 0.06288.
[c]Significant at 0.01% level.
[d]Significant at 1% level.

TABLE C

VALUES OF A′ OBTAINED FROM EXPERIMENT 2[a]

Comb. No.	Value of A′								
	50 Psi			100 Psi			200 Psi		
	4.5% A.C.	5.0% A.C.	5.5% A.C.	4.5% A.C.	5.0% A.C.	5.5% A.C.	4.5% A.C.	5.0% A.C.	5.5% A.C.
1	239	200	77	120	90	104	98	86	82
	253	150	87	134	105	124	114	73	78
	333	161	98	103	113	116	117	97	120
2	291	195	155	259	164	155	171	118	132
	420	204	127	280	151	211	160	76	92
	396	217	147	234	107	149	172	97	125
3	666	336	306	-	336	402	266	315	387
	867	669	303	747	280	420	315	315	494
	624	315	354	600	308	504	290	321	526
4	1,245	489	336	672	381	345	912	370	339
	442	342	336	872	687	366	1,050	277	453
	1,026	570	306	-	513	546	645	479	432

[a]Test temperature, 140 F; rate of loading, 30 gm/sec; height, 2.00 in.; A′ = 30 × A × 10^8 /gm-sec.

TABLE D

VALUES OF HEIGHT, H, AND A′ FROM EXPERIMENT 3[a]

Comb. No.	50 Psi				100 Psi			
	4.5% A.C.		5.0% A.C.		4.5% A.C.		5.0% A.C.	
	H (in.)	A′	H (in.)	A′	H (in.)	A′	H (in.)	A′
1	1.57	645	1.55	834	1.49	134	1.51	223
		858		834		140		309
		1,008		650		146		400
	1.81	693	1.75	200	1.75	100	1.74	134
		495		230		100		191
		588		220		102		130
	2.01	239	2.01	209	1.97	121	1.96	90
		253		150		134		105
		333		161		103		113
	2.25	348	2.25	129	2.21	31	2.21	57
		180		258		37		59
		182		122		37		59
	b = -3.906		b = -3.940		b = -4.532		b = -4.233	
	r^2 = 0.937		r^2 = 0.759		r^2 = 0.842		r^2 = 0.993	
2	1.54	810	1.54	594	1.52	237	1.51	174
		830		792		172		200
		822		498		214		207
	1.79	327	1.81	439	1.75	103	1.73	174
		514		432		110		124
		420		423		126		172
	2.02	291	2.01	195	1.99	259	1.98	163
		420		204		280		151
		396		217		234		107
	2.26	426	2.22	148	2.22	54	2.22	56
		220		146		40		50
		258		150		44		46
	b = -2.620		b = -4.120		b = -3.949		b = -3.142	
	r^2 = 0.916		r^2 = 0.951		r^2 = 0.994		r^2 = 0.771	
3	1.55	2,140	1.54	1,410	1.53	1,293	1.50	1,293
		2,200		1,410		1,500		537
		2,100		1,460		1,245		933
	1.79	1,540	1.81	1,300	1.79	468	1.75	498
		1,528		1,580		867		696
		1,540		756		936		591
	1.99	666	1.99	336	2.01	1,200	1.96	336
		867		670		747		280
		625		315		600		308
	2.24	360	2.22	177	2.28	379	2.23	205
		528		468		291		252
		730		220		-		312
	b = -4.109		b = -4.932		b = -3.131		b = -3.320	
	r^2 = 0.952		r^2 = 0.882		r^2 = 0.842		r^2 = 0.954	
4	1.54	1,720	1.54	1,850	1.53	1,135	1.52	1,310
		2,500		1,550		1,320		1,400
		2,245		1,720		1,515		1,470
	1.79	1,340	1.79	707	1.78	1,092	1.78	758
		1,340		1,101		864		780
		1,340		431		978		582
	2.01	1,245	2.01	490	1.98	672	1.98	381
		942		342		870		687
		1,003		570		-		513
	2.24	1,420	2.23	342	2.27	420	2.26	357
		417		368		471		357
		657		378		504		513
	b = -2.064		b = -4.228		b = -2.623		b = -3.140	
	r^2 = 0.992		r^2 = 0.959		r^2 = 0.971		r^2 = 0.961	

[a]Test temperature, 140 F; rate of loading, 30 gm/sec; $A' = 30 \times A \times 10^8$/gm-sec; $\log A' = \log a_o + b \log H$.

Traffic Simulator for Checking Mix Behavior

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Some asphaltic concrete mixes designed by accepted stability procedures have shown signs of distress under heavy volumes and loadings of traffic. In an effort to ascertain the effect of traffic on specimens of known stability, a laboratory traffic simulator was designed and built.

The specimens tested are the same as those used in either the Marshall or Hveem stability tests. The specimen is placed in a collar equipped with heating coils and temperature sensors, which in turn is boxed in the frame of the instrument in such a manner as to permit the traffic simulator wheel to travel over the center of the upper surface of the specimen. The temperature of the specimens may be maintained at any desired point, during test, by either the heater coils on the specimen collar or dry ice packed in the box. Six specimens may be tested simultaneously.

Tests conducted with the traffic simulator indicated that some mixes which meet stability criteria show distress under the test, other mixes having lesser stabilities showed less displacement than similar mixes having higher stabilities, also other mixes which resist displacement for 2,000 to 3,000 passes of the traffic wheel will suddenly show distress with a few hundred additional passes.

The only purpose of the traffic simulator is to serve as a supplementary check on the behavior of mixes under a moving wheel load.

•RECENTLY some asphalt pavements designed under accepted stability test methods and criteria have shown a tendency to distress under heavy volumes of traffic. The distress occurs in the form of rutting or channelizing in the wheel track area and on occasion as slight rippling or washboarding in areas of traffic acceleration or deceleration. Because the mixes were designed by recognized standard methods, produced and laid under exacting controls, and on check tests possessed stabilities well above minimum criteria, it appears that stability in itself is not always adequate to assure the desired service performance under heavy traffic.

The Bituminous Research Laboratory undertook the development of a device by which the trafficability or resistance of an asphalt mix to displacement under traffic could be evaluated and used as a check on design by stability procedures.

The instrument had to meet the basic requirements: (a) that it be compact enough to be used in a laboratory in conjunction with stability test procedures; (b) that it simulate the effects of traffic moving in one direction as nearly as possible; (c) that climatic conditions such as temperature and weather could be imposed on the mix

Paper sponsored by Committee on Mechanical Properties of Bituminous Paving Mixtures.

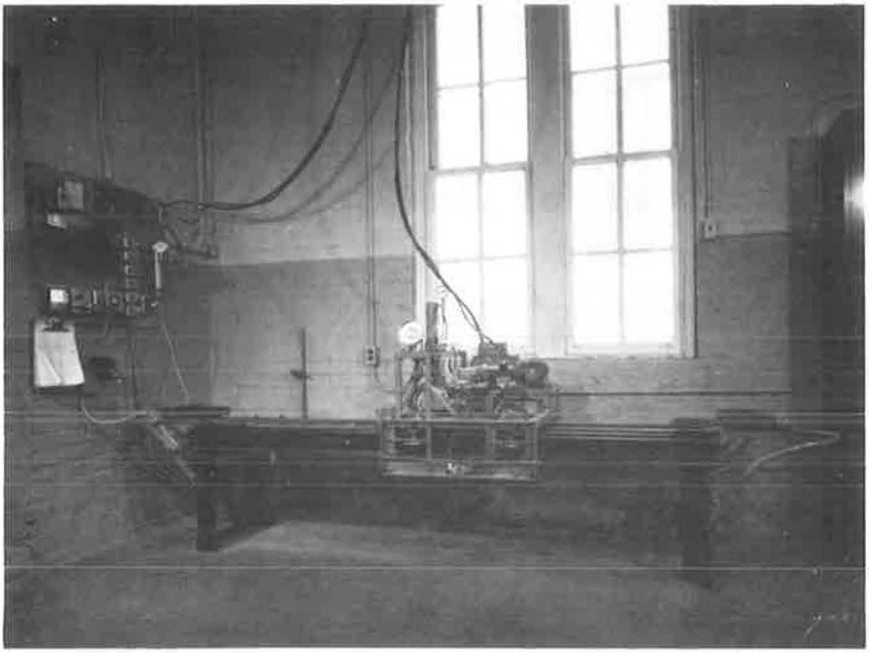

Figure 1. Traffic simulator.

under test; (d) that the extent and form of displacement under various volumes of traffic could be measured fairly accurately; (e) that the evaluation of trafficability could be related to the stability of the mix; and (f) that the trafficability of several mixes could be compared simultaneously.

Many ideas, concepts and theories of operation and controls, and tentative designs of the instrument were studied and investigated before development of the traffic simulator described herein. Because many adjustments and modifications were made as construction progressed, no detailed plans or drawings are available of the device.

Because the relation between the trafficability of a mix and its stability was of prime importance, it was decided that test specimens identical to those used in the Marshall and Hveem stability tests would be used in this test. Thereby the compaction variable would be eliminated and test results could be directly related to the method of stability test.

The manner of imposing the moving wheel load on the test specimen was resolved in favor of an oscillating carriage which is driven during its forward motion by the wheel imposing the load on the test specimens and during the reverse motion by an auxiliary drive. During the reverse motion the loading wheel is retracted, thus simulating only one-way traffic over the test specimens.

The traffic simulator is shown in Figure 1. It consists of a main frame, supported on legs bolted to the floor, in which the test specimen retainer box is installed and on which the carriage travels; a carriage which contains the loading wheel and other drive and control appurtenances; and a main operating control panel. Electrical power and compressed air are conveyed from the control panel to the carriage by the cable and hose suspended overhead.

The main frame is about 11 ft long, 3 ft wide and stands about 2 ft above the floor. It is composed of steel angles and channels so arranged as to provide tracks for the forward and backward movements of the carriage and to support the test specimen retainer box. The frame is also equipped with a group of thrust springs at each end which absorb the thrust of the carriage at the end of each run and which on release give the carriage an initial push on the reverse run. An electrical counter is mounted at the front end to total the number of forward passes made by the carriage in the course

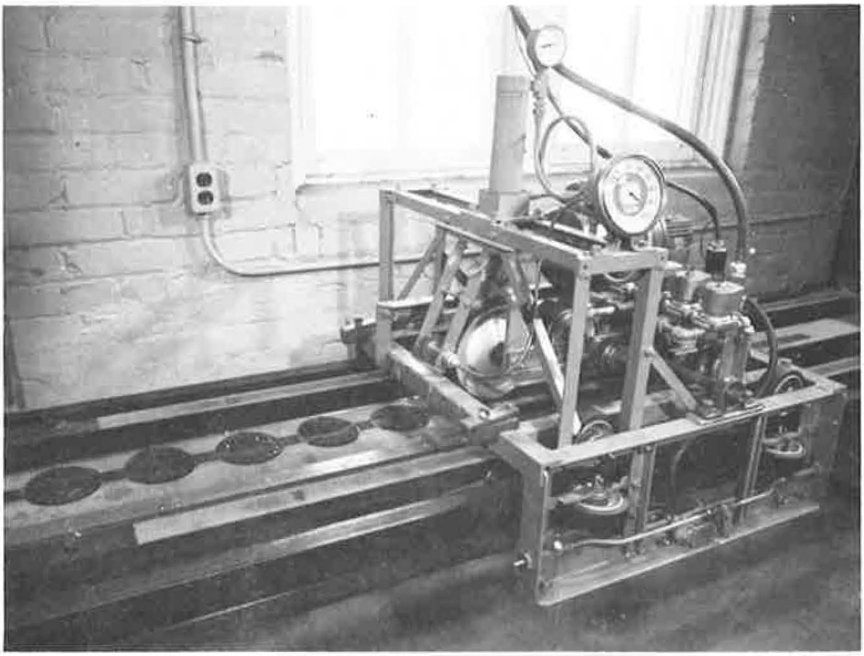

Figure 2. Traffic simulator, carriage and test specimen retainer box.

of a test. Adjustable spring load limit switch stops are provided at each end for the actuation of controls mounted on the carriage. The tracks on which the drive wheels of the carriage operate are surfaced with an abrasive for improved traction. At the forward end of the frame, space is provided for the installation of a removable test specimen retainer box in such a manner that the upper surface of the box is level with the loading wheel track and so that the loading wheel travels across the center of the surface of the test specimens (Fig. 2).

The carriage is designed to accommodate the loading wheel and the driving and control mechanisms necessary to operate it in an oscillating, to and fro, motion on the main frame. The carriage is equipped with two 4-in. diameter soft, solid rubber-tired, free-turning wheels on each upper side of the frame and four similar wheels on the underside of the frame to prevent the carriage from bucking during operation. Similar wheels are mounted, two on each side of the carriage, engaging the sides of the frame to prevent swaying (Figs. 2 and 3). A longitudinal rocker arm is mounted about centrally in the carriage. At its forward end is the loading wheel, 8 in. in diameter and $1\frac{1}{2}$ in. wide and fitted with a $1\frac{1}{8}$-in. wide soft, solid, rubber tire. At its rear, the rocker arm has two 5-in. diameter soft, solid, rubber-tired, reverse-drive wheels. The rocker arm is actuated by a 2-in. diameter vertical, double-acting, compressed-air ram which raises and lowers its front end to engage loading and reverse-drive wheels and also applies the desired pressure on the loading wheel. The loading wheel usually applies a load of about 80 psi to the test specimens, but this may be adjusted by varying the air pressure to the ram. The ram is operated and controlled by a limit switch mounted on the side of the carriage which actuates a number of electric solenoid air valves feeding air to the ram. The carriage is driven by a $\frac{1}{2}$-hp motor mounted thereon and connected to the loading and reversing wheels through a reducing gear and belt and gear drives in such a manner that the wheels rotate in their respective directions continually. The travel speed of the carriage is about 4 mph, thus permitting about 1,400 passes of the loading wheel over the test specimens per hour. The travel speed may be somewhat varied by changes in drive gear or belt ratios. A tachometer attached to the loading wheel indicates travel speed. Holding clamps are provided at the front end of the carriage for the displacement measuring gage (Fig. 3). A small water tank

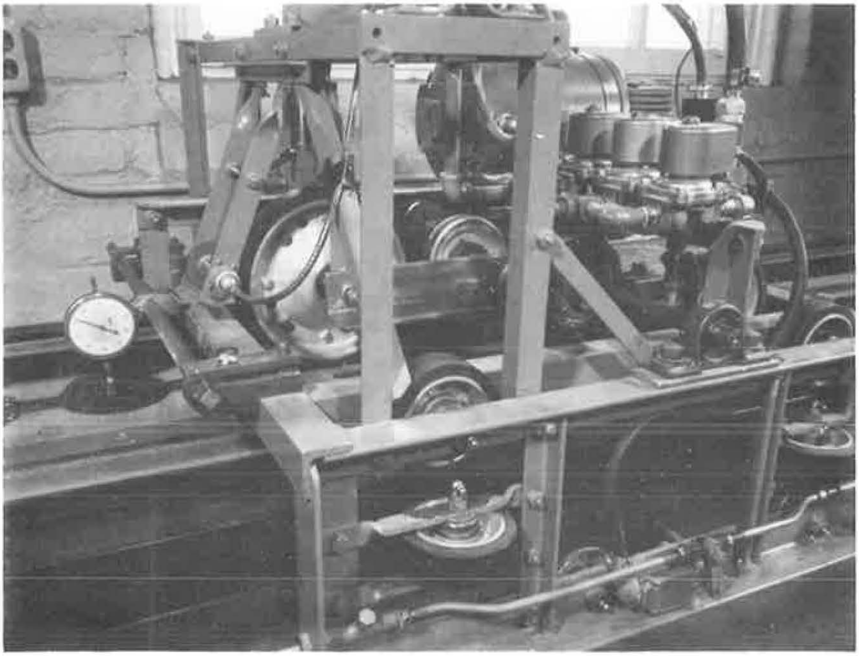

Figure 3. Traffic simulator carriage and displacement measuring gage.

operated by compressed air furnishes water to a fine water spray nozzle, simulating rain on the surface of the test specimens.

In starting, with power off, the carriage is manually pulled to the rear of the frame. The limit switch is then actuated so that when the power and compressed air are turned on, the ram lowers the rotating loading wheel, engaging its track to move the carriage forward. When the carriage reaches the front, several operations occur simultaneously. The thrust springs bring the carriage to a stop; the limit switch actuated by the stop raises the loading wheel and engages the reversing wheels, and the thrust springs release, starting the carriage in the reverse direction, thus reducing the strain on the reverse drive. When the carriage reaches the other end, a similar action takes place in which the loading wheel is engaged and the carriage moves forward again. By proper adjustment of the limit switch stops at the ends of the frame, a smooth forward and backward movement of the carriage may be secured.

The specimen retainer box is made of $\frac{3}{8}$-in. steel plates forming a box 36 in. long, 8 in. wide and $3\frac{1}{4}$ in. high. It is fitted with a $\frac{1}{4}$-in. thick steel cover in which $4\frac{1}{4}$-in. holes have been bored to hold and align the specimen holding rings in position centrally along the line of travel of the loading wheel on $5\frac{1}{2}$-in. centers. Six test specimens may be tested simultaneoulsy by this arrangement. In the bottom of the box under each holding ring position, three screws are provided for adjusting the surface of the test specimen level with the top of the retainer box.

The specimen holding ring is a steel tube $2\frac{3}{4}$-in. long machined to an outside diameter of $4\frac{1}{2}$ in. and an inside diameter of just slightly more than 4 in. so that a Marshall or Hveem stability test specimen 4 in. in diameter may be inserted tightly. The ring has a machined $\frac{1}{8}$-in. shoulder $\frac{1}{4}$ in. long around its top (Fig. 4). The shoulder fits into the hole in the cover of the box, thus aligning and locking the rings in position as the cover is bolted down. At the bottom of each ring is a $\frac{1}{8}$-in. thick circular steel plate. This plate serves as a solid base for the specimen and as a seat against which the screws in the bottom of the box act in adjusting the top of the specimen level with the top of the box. About 1 in. from the top of the ring a $\frac{1}{8}$-in. hole is provided through its side for insertion of a thermocouple to measure the temperature of the specimen during test. Each ring is also provided with a flexible heating tape, 1 in. wide and 24

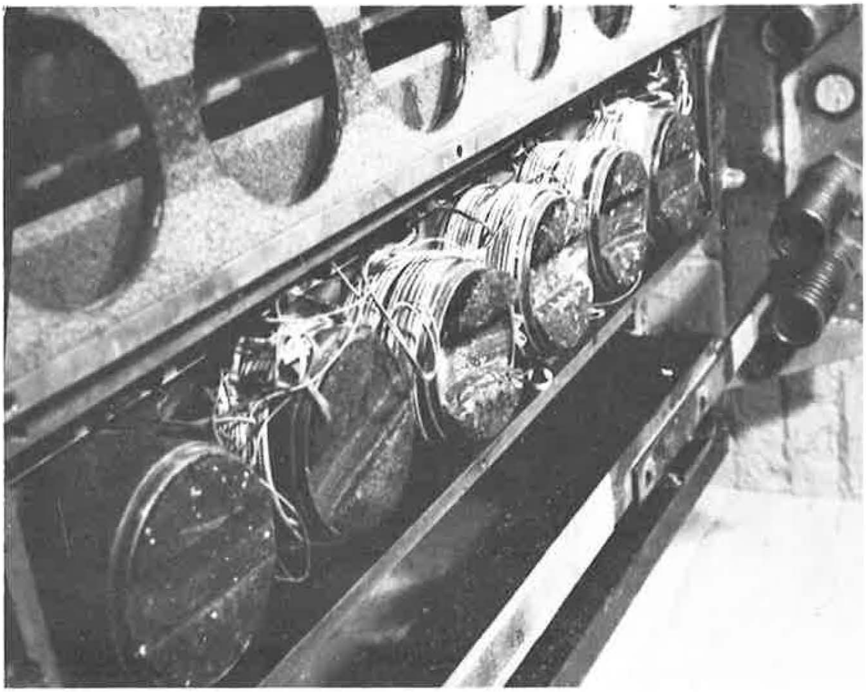

Figure 4. Assembly of specimen retainer box.

in. long, operating at 110 and about 200 w, which is wound around the outside of the ring for heating the specimens (Fig. 4). Plug-in electrical connections for both the heaters and thermocouples are provided inside the box. Specimen temperatures up to 200 F may be secured by this heating system. If cooling of specimens is desired, the inside of the box around the holding rings may be packed with dry ice.

The traffic simulator is operated from a main control panel mounted on the wall beside the instrument (Fig. 5). The electrical power controls, mounted across the top of the panel, consist of a main power switch, thermal overload relay, toggle switch for automatic controls on the carriage, drive motor start-stop switch, and a plug-in outlet for connection of overhead power cable to the carriage. Compressed air at 150 psi is piped to an air pressure regulator valve and gage, mounted on the right side of the panel, from which compressed air at the desired pressure is conveyed to the carriage by the overhead flexible pressure air hose. Specimen heating is controlled by a 15-amp, 0- to 240-v variac with telltale voltmeter and individual toggle switches connecting to the heater tapes on the respective holding rings. These controls are mounted in the center of the panel. The thermocouples inserted in each of the specimens are connected to a dial pyrometer through a selector switch. Thus, the temperature of any one of the specimens may be read at any instant. These instruments are mounted at the lower left-hand section of the panel and are connected to the specimen retainer box by plug-in multiple wire cables.

The temperature of the specimens is controlled manually in the following manner. The variac is set at 115 v and the individual switches are turned on. The temperature of each of the specimens is read periodically noting the rate of rise of temperature. As the temperature approaches the test temperature, the voltage on the variac is reduced so that the desired temperature is maintained. Temperature may also be maintained independently on individual specimens by operation of their respective control switches. In this manner specimens may be tested at different temperatures.

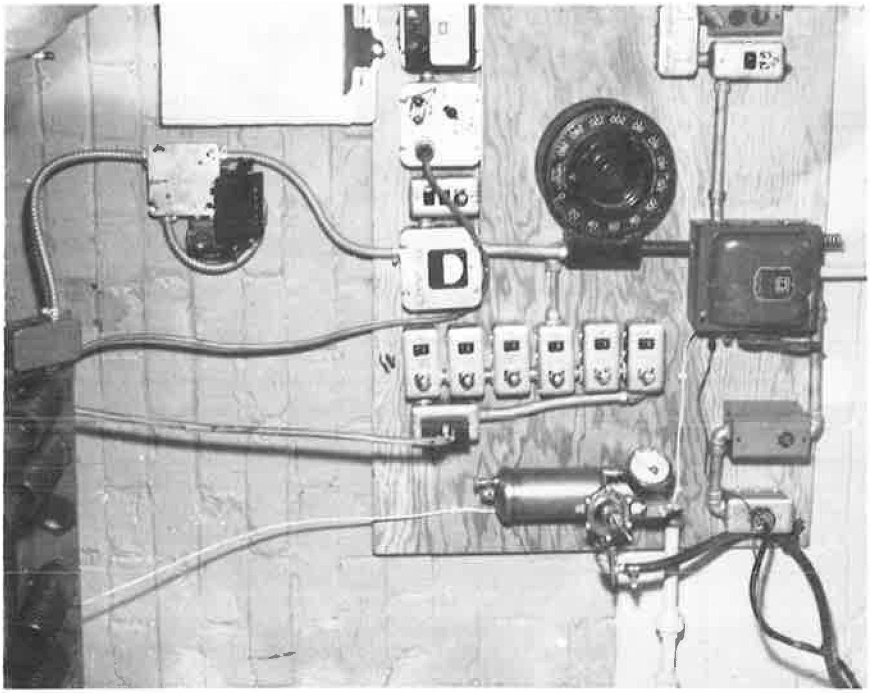

Figure 5. Control panel.

TRAFFIC SIMULATOR TEST PROCEDURE

Because the traffic simulator test was devised primarily as a check on behavior of mixes under a moving load, it is desirable that at least two specimens of mixes compacted by standard procedure for test by Marshall or Hveem stability be made available.

The compacted test specimen is inserted into a holding ring over the base plate with its top surface flush with or slightly below the top of the ring. If the specimen fits loosely in the ring, metal shims are inserted around its side to tighten it. A small hole is bored into the specimen through the thermocouple hole in the ring and the thermocouple is inserted. The holding ring is placed into the retainer box over the adjusting screws. When all holding rings containing specimens, or dummies if sufficient number of specimens are not available, are in position in the box, the box cover is put in place with the tops of the rings inserted in the holes therein. The cover is then bolted down onto the box, thus bringing the rings into proper alignment. Each specimen is then adjusted to bring its top surface level with the top of the box by the adjustment screws.

As soon as the specimens are in position, the heaters are turned on and the specimens are brought to the desired test temperature. Tests have been conducted at room temperature, and at 100, 140 and 160 F; however, 100 F has been arbitrarily selected as the test temperature for most so far conducted. About 20 to 30 min are needed to raise and adjust controls to maintain specimen temperature at this level. When specimens have attained the test temperature, the surfaces of the specimens are again checked for level with the top of the box and readjusted if necessary.

The Ames dial displacement gage, reading to 0.001 of an inch, is attached to the carriage. The carriage is then moved over a bench mark on the forward end of the frame and the displacement gage is set at zero. The gage is then moved on its support to the center line of loading wheel travel and locked into position. Gage readings are taken on the surface at back, center, and front of each specimen. The points where measurements were taken are marked on the specimen and beside the specimen on the box cover to assure that subsequent measurements are taken at the same points.

At the beginning of the test, the counter is set for 300 passes and the compressed air pressure is set for desired loading on the loading wheel. Forty pounds pressure has been used in most tests conducted because it provides an equivalent of 80-psi tire loadings on the specimens. The main power switch is then turned on, the automatic controls on the carriage are actuated and the drive start button is pushed; the carriage moves forward and backward automatically. Specimen temperatures are checked periodically with the pyrometer. After 300 passes have been completed, the counter automatically stops the carriage.

The displacement gage is again attached to the carriage, the bench mark is checked and readings are taken at the same points on the specimens as before. The initial 300 passes set the specimens in their holders. If desired, the readings at this time may be taken as the data for further displacement in preference to the original readings. If the specimens tend to rotate in the holders, they should be wedged into position by side shims. After the readings have been taken, the gage is removed, the counter is set for a total of 1,000 passes and the test is continued. Displacement readings are taken and recorded after each successive 1,000 passes. The test may be continued for any number of passes desired or may be discontinued when specimens show excessive displacement or distress. Usually the tests have been conducted for 5,000 passes.

On completion of the test, total displacement at each of the three points is calculated for 1,000, 2,000, 3,000, 4,000, and 5,000 passes. Thus, mixes may be evaluated or compared on the basis of the displacement occurring after a certain number of passes or of the number of passes required to create a predetermined amount of displacement. Generally, comparisons have been made on the basis of total displacement occurring at the center of the specimen at 5,000 passes. Mixes that show a displacement of more than $\frac{1}{8}$ in. at or before 5,000 passes are arbitrarily deemed suspect to distress under traffic. Displacements occurring at the front or rear of the specimens in the form of excessive depression or heave are indicative of a tendency of the mix to shove. Such results must be evaluated independently.

On removal from the rings after completion of the test, the specimens may be subjected to further observation and test. The specimens may be sawed in sections by a diamond saw and the effects of traffic upon orientation of particles, degradation of particles, absorption of asphalt by particles, densification of mix and other features may be observed and tested.

TEST RESULTS

Mixes of different compositions behave in various ways in the traffic simulator test. Most mixes having a fairly good resistance to displacement under a moving load displace in the test in a shape similar to a catenary wherein the greatest displacement occurs in the center of the specimen and but little at the ends (Fig. 6). In such cases the material appears to be pushed to the sides of the wheelpath. Other mixes of lesser resistance tend to displace sharply at the rear of the specimen with displaced material pushed ahead into a heaved section at the forward end. Some mixes resist displacement for 30,000 to 50,000 passes, whereas others displace excessively within 1,000 to 2,000 passes. Others resist displacement for 3,000 to 4,000 passes and then fail suddenly after a few hundred additional passes. Still others displace somewhat in the first 1,000 to 2,000 passes and then resist further displacement up to and beyond 5,000 passes. Temperature appears to have a material effect on behavior. Some mixes resisting displacement at 100 F up to 5,000 passes show excessive displacement within a few thousand passes when the temperature is raised to 140 F. This is particularly true of mixes with higher asphalt contents. Generally, the amount of displacement at 140 F is greater than at 100 F for the same mixes.

Test specimens cut into sections on conclusion of the test exhibit some interesting results. A definite orientation of coarser aggregate particles can be noted in mixes having higher asphalt contents and larger quantities of fine aggregates. Degradation of particles in dryer mixes can also be observed. Excessive absorption of asphalt by

TABLE 1

TRAFFIC SIMULATOR TEST RESULTS

Filler (%)	A. C. (%)	Displacement[a] (0.001 in.) No. of Passes 1,000	3,000	5,000	Marshall Stab. 50 Blows	Displacement[b] (0.001 in.) No. of Passes 1,000	3,000	5,000	Hveem Stab. Dry	Iowa Stab. Wet
(a) Ocheydan Aggregate										
Lime:										
0.0	4.1	47	60	70	470	57	67	NR	34	27
	4.6	55	90	110	927	62	75	NR	53	19
	5.1	120	192	400	450	130	137	140	32	26
2.5	4.1	85	85	100	1,725	90	91	NR	38	39
	5.1	102	157	185	1,925	130	160	178	35	35
	6.1	109	140	162	1,470	74	74	81	47	27
5.0	5.0	90	118	118	937	55	92	115	40	28
	6.0	104	128	139	970	10	30	51	52	28
	6.9	79	160	189	1,875	60	90	112	40	26
7.0	4.9	72	90	100	2,045	25	40	40	44	32
	5.8	78	95	131	1,120	4	12	21	46	36
	6.7	122	240	NR	1,500	120	145	155	35	47
10.0	5.2	70	90	106	2,373	70	80	123	47	24
	6.2	127	187	230	1,308	175	268	NR	46	45
	7.0	485	NR	NR	1,875	477	NR	NR	40	26
Loess:										
2.5	4.0	125	210	NR	488	120	140	NR	40	43
	5.0	95	110	110	815	46	70	87	40	41
	6.0	122	153	160	803	50	60	60	46	36
5.0	5.0	68	88	110	1,600	40	60	69	42	36
	6.0	122	149	181	1,440	10	29	40	41	26
	7.0	443	NR	NR	1,335	50	NR	79	36	42
7.0	5.0	95	100	211	1,030	NR	NR	NR	47	34
	6.0	75	75	204	1,315	6	10	10	62	25
	7.0	135	158	165	1,263	28	28	28	36	42
10.0	6.0	70	72	90	1,780	95	100	100	52	23
	7.0	175	240	NR	1,170	NR	NR	NR	50	36
	8.0	317	NR	NR	1,480	41	70	75	44	43
(b) Garner Aggregate										
Lime:										
0.0	4.2	NR	NR	NR	190	98	115	NR	44	28
	4.7	70	120	140	430	139	160	175	32	35
	5.1	135	185	192	310	111	140	151	43	32
2.5	4.2	80	130	145	1,045	115	140	NR	44	35
	5.2	60	90	110	1,410	87	110	115	46	36
	6.1	130	180	207	1,215	25	68	100	51	27
5.0	5.0	360	NR	NR	710	30	129	130	53	23
	6.0	148	248	NR	1,230	97	110	110	54	30
	7.0	571	NR	NR	1,060	507	NR	NR	51	49
7.0	5.0	50	59	72	1,730	58	65	88	42	28
	6.0	70	80	204	1,375	77	90	110	48	30
	7.0	125	310	NR	965	180	283	NR	37	55
10.0	5.0	48	68	70	1,790	95	120	127	50	26
	6.0	85	100	120	1,255	160	160	185	53	28
	6.8	104	130	145	1,630	155	250	NR	52	23
Loess:										
2.5	4.0	NR	NR	NR	1,245	70	80	NR	52	27
	5.0	80	80	95	1,275	50	85	100	54	27
	6.0	50	70	75	1,240	30	37	37	48	25
5.0	5.0	80	85	103	1,340	85	105	105	42	43
	6.0	123	142	181	1,170	0	15	25	39	45
	7.0	135	271	NR	1,080	95	130	149	50	49
7.0	5.0	31	40	40	1,505	57	63	63	46	30
	6.0	102	125	139	1,080	80	80	80	47	30
	7.0	139	170	190	1,550	119	150	185	32	65
10.0	5.0	70	75	78	1,690	70	72	72	68	60
	6.0	50	55	65	1,460	40	40	40	41	68
	7.0	90	90	90	2,025	35	40	40	38	49
	8.0	157	165	185	1,585	25	40	56	28	63
(c) Columbus Junction Aggregate										
Lime:										
0.0	4.0	25	63	110	820	5	15	25	NR	42
	4.5	9	9	40	660	20	33	40	43	41
	5.0	25	40	40	730	35	40	40	35	47
2.5	4.0	5	11	28	750	16	25	25	41	44
	4.8	8	10	18	870	3	20	20	40	47
	6.0	8	8	20	810	5	10	35	41	47
5.0	4.9	0	23	30	1,028	25	25	30	39	47
	6.1	33	37	50	960	25	25	25	43	52
	7.0	23	40	52	890	15	35	40	35	52
7.5	5.1	18	25	35	1,410	20	40	48	40	44
	6.0	22	70	87	1,230	9	30	40	41	51
	6.9	15	23	45	1,240	20	40	57	37	53
10.0	6.0	17	45	55	1,510	15	15	45	41	46
	7.1	23	50	63	1,310	35	41	53	29	67
	8.2	10	40	53	657	65	105	110	NR	NR
Loess:										
2.5	4.0	10	35	39	1,060	11	15	15	33	41
	5.0	3	15	17	1,050	25	30	30	38	43
	6.0	3	10	10	1,090	11	15	25	40	47
7.0	5.0	0	8	10	1,380	3	3	7	39	47
	6.0	9	12	15	1,550	10	10	18	47	43
	7.0	10	20	25	1,500	14	14	18	34	53
10.0	5.0	NR	NR	NR	1,710	5	9	9	44	36
	6.0	10	10	10	1,500	10	10	10	43	67
	7.0	6	10	21	1,050	3	5	10	39	73

[a]Samples dynamically compacted. [b]Samples statically compacted.

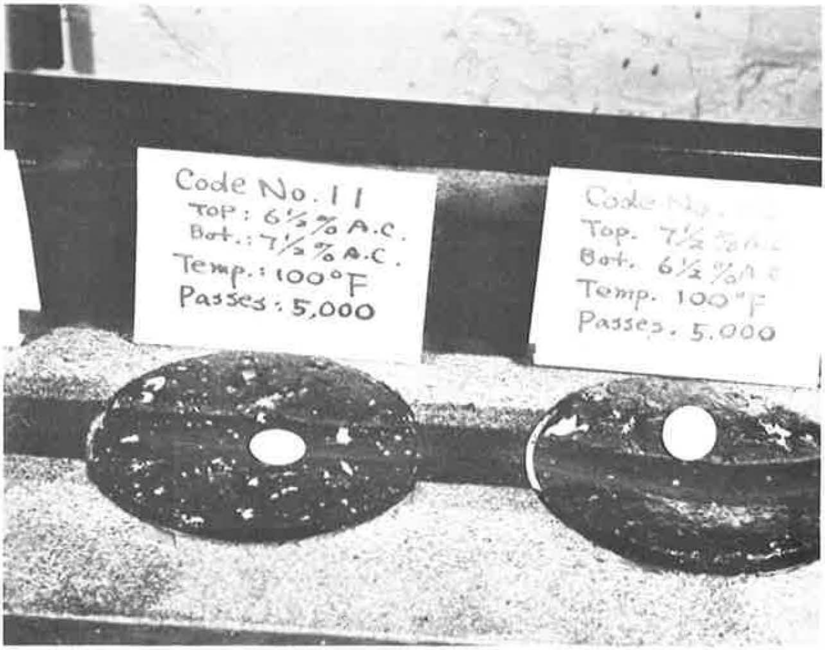

Figure 6. Displacement of mix after test.

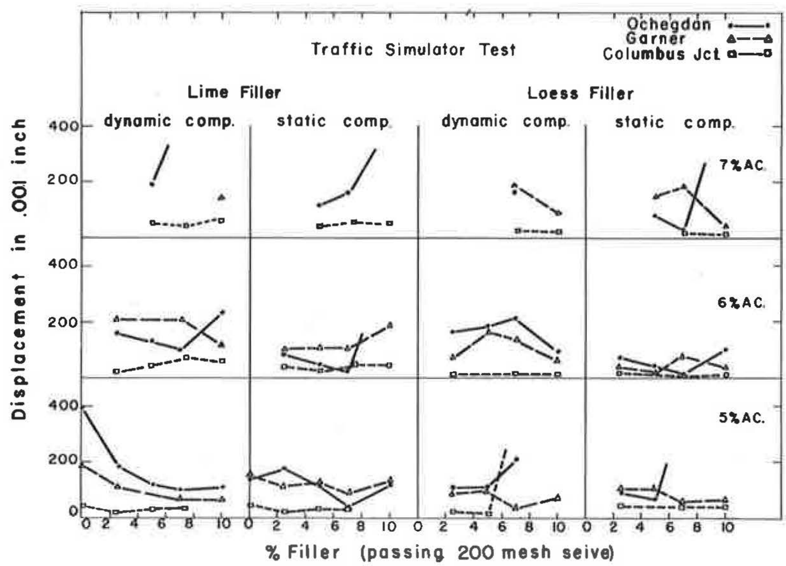

Figure 7. Displacement of samples at 5,000 passes.

aggregate particles is clearly apparent in mixes containing absorptive or relatively porous aggregates. Densification of the mix under the wheelpath is also quite noticeable. By proper sectioning of the test specimen, variations in density and asphalt content in several sections may be determined by test.

The results of an investigation conducted at the Bituminous Research Laboratory are presented here as an example of the type and character of results obtained in the traffic simulator test. The mixes in that investigation contained three different types of aggregates and two different types of fillers. Details of physical properties of materials, and mix proportions, are given by Csanyi et al. (1). The three aggregates used were a partially crushed gravel, designated as Ocheydan; a hard crushed limestone, called Garner; and a softer crushed limestone, referred to as Columbus Junction. The fillers used were a limestone filler and a pulverized loess filler. Mixes of each of the fillers containing a range of asphalt contents were tested for trafficability, Marshall, Hveem and Iowa stabilities. The results of these tests are given in Table 1, and the trafficability results at 5,000 passes are plotted in Figure 7. The notation NR, indicating no definite results, means that a displacement in excess of ½ in. was secured at that point in the traffic simulator test.

DISCUSSION OF RESULTS

The trafficability results at 5,000 passes (Fig. 7) indicate that the behavior of mixes containing various combinations and proportions of materials can be evaluated and compared as to their resistance to displacement under a moving load by the traffic simulator. The evaluation or comparison may also be based on the type of aggregate, type and quantity of filler or asphalt, methods of compaction, and specimen densities. The test also discloses the rate of distress with relation to number of passes and temperature.

The results also show that no direct relationship exists between trafficability of a mix and its stability, regardless of the method used to determine stability. Mixes with relatively high stabilities, particularly those with the higher asphalt contents, tend to fail, whereas mixes with lower stabilities stand up quite well under the traffic simulator test.

The traffic simulator test has been compared with field service behavior in only two instances. In both of these cases the mixes being laid possessed more than adequate stabilities and met all other criteria for a high-type pavement. On test in the traffic simulator these same mixes indicated a tendency to distress under a moving load. Within a year in service both pavements began to show some distress in the form of rutting and channeling in the wheel track areas. This experience confirmed that mixes designed in accordance with accepted criteria are not always adequate to meet the stresses of heavy traffic and indicated that some form of additional check, such as the traffic simulator, capable of denoting a tendency to distress, is needed if the desired service behavior is to be assured. Several other service comparisons are now in progress.

No attempt has yet been made to establish definite criteria for this test by which the suitability of a mix for various volumes or types of traffic may be assessed. However, based on experiences with mixes known to perform satisfactorily and on the behavior of the mix during test, certain specific test conditions have been adopted. These include a specimen temperature of 100 F during test, an applied wheel load of 80 psi, and an arbitrary maximum displacement of ⅛ in. after 5,000 passes as an indication of the ability of the mix to resist the effects of heavy traffic volumes. There is no assurance that these conditions are correct or that test results obtained under them are directly related to service behavior. Only further correlation between traffic simulator results and service records can establish their reliability or indicate the adjustments needed to develop dependable criteria for the instrument.

However the test results indicate that, under the conditions adopted, the traffic simulator may be used as a supplementary check of the behavior of a mix under a moving load. It may also be employed as a research tool to investigate the effects of simulated traffic on the internal structure and behavior of a mix.

REFERENCE

1. Csanyi, L. H., Cox, R. E., and Teagle, C. R., "Effect of Fillers on Concrete Mixes." Highway Res. Record 51 (1964).

Effect of Fillers on Asphaltic Concrete Mixes

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This study, attempts to determine the effects on a filler on the properties of asphaltic concrete mixes. Three different types of aggregates, crushed gravel, crushed hard limestone, and crushed soft limestone, were used in conjunction with two different types of fillers, crushed limestone dust and pulverized loess, in the preparation of a series of asphaltic concrete mixes. The mixes contained 0, 2½, 5, 7 and 10 percent minus 200-mesh filler and asphalt contents from 4 to 8 percent of an 85 to 100 penetration asphalt cement. The mixes were tested for Marshall, Hveem and Iowa stabilities; Hveem cohesion; voids in compacted specimens; and trafficability under a moving load in a traffic simulator. A supplementary series of tests was also made to ascertain the effects of small quantities of asbestos added to some of these mixes.

The results of the study indicate (a) that quantity type of filler used in conjunction with fixed quantities of binder have a material effect on the properties of mixes; (b) that the type of aggregate used in mixes containing the same quantities of filler and of binder has its effect on mix properties; (c) that some methods of test for properties of mixes are more sensitive to the effects of fillers than others; (d) that stability alone is not always a dependable criterion in determining mix behavior under moving loads; and (e) that certain quantities of asbestos in certain mixes have a beneficial effect on properties of mixes tested under moving loads.

THE EFFECT of fillers on the properties and behavior of hot-mixed asphaltic concrete mixes prepared by conventional mixing procedures has been fairly extensively studied in the past. Usually the studies have been limited to a specific type of filler with a specific type of aggregate. The purpose apparently has been to determine the optimum quantity of filler for the best results as far as the physical properties and service behavior of the mixes were concerned. The types, characteristics, and properties of the fillers, aggregates, and asphaltic binders varied widely in these studies. It has therefore been difficult to develop theories concerning the function and specific effects of fillers and to apply these theories to the behavior of specific mixes.

Two rather general theories, however, have emerged from the studies. One suggests that the filler, an inert mineral of relatively very fine particle size distribution, serves primarily to fill the voids in the fine aggregate or coarse and fine aggregate combinations. It thereby increases the density of the resulting compacted mix. Another theory suggests not only that the filler fills voids in the aggregate but also that some of the filler hardens and toughens the binder, making a denser, tougher mix because a portion of the fraction of the filler passing the 325-mesh sieve becomes suspended in the asphaltic binder.

The first therory is used extensively in designing asphaltic mixes to control aggregate blending to obtain the desired densities. Filler contents are kept rather low to avoid brittleness and cracking of mixes in service. Some of the minute particles of the filler enter the asphalt, particularly when mixes are prepared with softer asphalts

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and at the upper allowable mixing temperature limits. The latter theory seems to be more typical of actual conditions and may increase in importance with the trend toward control of viscosity of the binder during mixing.

Some difficulties have been found in interpreting and evaluating the results of the studies because of differences in use of the term "filler." This term is often used loosely to designate a mix material with a particle size distribution smaller than the aggregates. Sometimes filler has a particle size distribution from retained on the No. 4 sieve to passing the 200-mesh sieve. At other times all the filler passes the 200-mesh sieve. Specifications often define a filler, regardless of basic origin or chemical composition, as an inert mineral flour or dust having a specified particle size distribution from passing the No. 10 sieve down to passing the 200-mesh sieve. The AASHO filler specifications are much more specific in that gradation is limited to specified quantities passing the No. 30 sieve and passing the No. 200 sieve (1). Some of the studies have included a gradation of the filler. Seldom, however, has that portion of material passing the No. 200 sieve which adheres to the aggregates or are combined within them, as determined by the wash test, been reported. In some instances, especially where softer aggregates are used, this portion may amount to an appreciable quantity. Because the total fraction of all materials passing the No. 200 sieve contained in a mix is known to have a decided effect on mix properties, such totals should be available if realistic evaluations are to be made.

In this study, therefore, the filler will be restricted to the fraction of the mix materials passing the No. 200 sieve, either as part of the aggregate or of the so-called mineral filler.

Purpose and Scope

The purpose of this study was to ascertain the effects on the physical properties of asphaltic concrete mixes of various types and quantities of filler in combination with various types of aggregates.

Separate series of tests were conducted to determine the effect of small quantities of asbestos added to the mixes with the Ocheydan and Garner aggregate and with the limestone filler.

All mixes were tested for Marshall, Hveem, and Iowa stabilities, and voids in compacted specimens and other pertinent data were determined. The test specimens were also subjected to a trafficability test in a laboratory traffic simulator.

Test of Materials

The materials used in the investigation, except the loess and asbestos were obtained from hot-mix Type A asphalt concrete paving projects in the vicinity of Ocheydan, Garner, and Columbus Junction, Iowa. In the project on Iowa Route 9 near Ocheydan, the coarse aggregate being used was crushed gravel; in the project on US 69 near Garner, it was a hard crushed limestone; and in the project on US 218 near Columbus Junction it was a soft limestone.

The fine aggregate was a sand, and the mineral filler was a limestone dust available locally in each area. The loess used as a filler in this investigation was obtained from Carroll County, Iowa, through the courtesy of County Engineer M. L. Schmeiser. The asphaltic binder was an 85 to 100 penetration asphalt cement, and the asbestos was grade 7M06, furnished by the Canadian Johns-Manville Co. To simplify identification, the materials are designated by job source i.e., Ocheydan, Garner, and Columbus Junction. The physical properties of the separate materials are given in Tables 1, 2, 3, and 4.

The damp loess as received from the field was dried in pans on a thermostatically controlled hot plate at 500 F for 24 hr. After

TABLE 1

PHYSICAL PROPERTIES OF ASPHALT CEMENTS

Source	Penetration[a] 77/100/5	Flash Pt.[b] (° F)	Soft. Pt.[c] (° C)	Spec. Grav.[d] at 77 F	Ductility[e] at 77 F
Ocheydan	90	637	50	1.02	150+
Garner	89	602	46	1.03	150+
Columbus Jct.	91	665	48	0.994	150+

[a]By ASTM D-5. [b]By ASTM D-92. [c]By ASTM E-28.
[d]By ASTM D-70. [e]By ASTM D-113.

TABLE 2

PHYSICAL PROPERTIES OF AGGREGATES

Aggregate	Total Percent Passing										Spec. Grav.[a]		L. A. Abrasion[b] (%)	V. M. A.[c] (%)	Unit Wt.[d] (pcf)
	3/4 in.	5/8 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 40	No. 80	No. 100	No. 200	Apparent	Bulk			
Ocheydan:															
3/4 in.	100.0	86.5	55.6	19.6	—	—	—	—	—	2.7	2.73	2.70	25.2	40.9	98.4
3/8 in.	—	—	—	100.0	46.8	5.5	—	—	—	1.9	2.70	2.66	35.2	40.8	95.3
Sand	—	—	—	—	100.0	94.7	33.5	15.5	13.4	8.3	2.74	2.71	—	21.0	127.6
Garner:															
3/4 in.	100.0	92.4	69.8	23.0	3.3	1.4	1.3	—	1.0	0.7	2.79	2.75	25.0	40.5	102.0
3/8 in.	—	—	—	100.0	47.0	3.5	—	—	2.2	1.2	2.78	2.76	27.0	39.6	104.0
Sand	—	—	—	—	100.0	93.0	51.0	18.5	13.2	5.9	2.78	2.74	—	32.7	115.0
Columbus Jct.:															
3/4 in.	100.0	—	57.2	—	3.9	2.0	1.9	1.8	—	1.6	2.66	2.48	28.9	43.3	87.7
3/8 in.	—	—	100.0	99.5	33.6	3.4	1.5	1.4	—	1.2	2.63	2.45	25.6	41.5	87.4
Sand	—	—	—	—	100.0	73.1	21.2	3.9	—	1.1	2.61	2.43	—	31.8	111.4

[a] By ASTM C-127. [b] By ASTM C-131. [c] By ASTM C-30. [d] By ASTM C-29.

TABLE 3

PHYSICAL PROPERTIES OF MINERAL FILLERS

Filler	Total Percent Passing										
	No. 4	No. 8	No. 40	No. 80	No. 100	No. 200	No. 270	No. 325	No. 400	5 μ	1 μ
Ocheydan[a]:											
Dry sieve	100.0	99.5	93.0	69.9	62.6	45.4	—	—	—	—	—
Hydrometer	—	—	—	—	—	—	36.0	32.0	28.0	2.0	—
Garner[b]:											
Dry sieve	100.0	100.0	98.5	84.0	76.5	48.5	—	—	—	—	—
Hydrometer	—	—	—	—	—	—	38.0	34.0	29.5	5.0	2.0
Columbus Jct.[c]:											
Dry sieve	100.0	100.0	90.2	57.9	48.8	27.3	—	—	—	—	—
Hydrometer	—	—	—	—	—	—	25.2	20.0	16.2	3.9	2.2
Carroll Co. loess[d]:											
Dry sieve	100.0	99.4	69.9	53.6	50.2	46.0	—	—	—	—	—
Hydrometer	—	—	—	—	—	—	99.0	96.5	94.0	90.0	24.5

[a] Spec. grav., 2.65. [b] Spec. grav., 2.74. [c] Spec. grav., 2.68. [d] Spec. grav., 2.74.

TABLE 4

PHYSICAL PROPERTIES OF CANADIAN CHRYSOTILE ASBESTOS 7M06

Properties	Values
Specific gravity	2.55
Fiber diameter (in.)	0.000000706 to 0.00000118
Fibrils in 1 in.	850,000 to 1,400,000
Tensile strength (psi)	100,000 to 355,000
Grading, Quebec standard test, minimum wt. (oz)	
1/2 mesh	0
4 mesh	0
10 mesh	1
Pan	15
Quebec standard classification	7M06 Medium bulk and absorption

heating, the loess was lumpy, hard, and dusty. The lumps when struck with a hammer shattered readily and appeared dry inside. The dried loess was then passed through a laboratory hammermill (Fig. 1) and converted to a mineral filler with less than 1 percent moisture and a particle size distribution as shown in Table 3. Raw loess can be converted to a mineral filler in quantities in excess of 25 tons per hour commerically and economically (2).

Composition of Mixes

To study the effect of fillers on the physical properties of mixes, mixes containing 0, 2.5, 5, 7, 7.5, and 10 percent filler were used. Job-mix formulas for each construction job using the coarse aggregates were obtained from the Iowa Highway Commission.

The percentages of each of the aggregates, the ¾- and ⅜-in. coarse aggregates and the sand given in the job-mix formula, were converted to weights to provide a 50-lb blend for these aggregates. The weights of the separate aggregates in the 50-lb blend were held constant for all mixes in each aggregate group. In preparing each of the mixes within a group, local mineral filler or loess was added to the fixed aggregate blend in quantities necessary to provide the minus No. 200 sieve filler content. The aggregates were washed to remove minus 200-mesh material for the zero percent filler content mixes. Aggregates were dry sieved to remove excess minus 200-mesh material for the 2.5 percent filler content mixes. The aggregate-filler composition for each mix was calculated on a basis of percentage of each mineral constituent by weight of

TABLE 5

PROPORTIONS OF MIX CONSTITUENTS USING VARIOUS AGGREGATES

Ocheydan						Garner						Columbus Junction					
-200[a] (%)	Aggregate[b] (%)				Asphalt[a] (%)	-200[a] (%)	Aggregate[b] (%)				Asphalt[a] (%)	-200[a] (%)	Aggregate[b] (%)				Asphalt[a] (%)
	3/4 in.	3/8 in.	Sand	Filler			3/4 in.	3/8 in.	Sand	Filler			3/4 in.	3/8 in.	Sand	Filler	
						(a) Lime Filler											
0[c]	34.41	23.65	41.94	0	4.12	0[c]	27.18	25.00	47.82	0	4.20	0[c]	35.00	30.00	35.00	0	4.0
					4.62						4.70						4.5
					5.10						5.10						5.0
2½[d]	34.41	23.65	41.94	0	4.12	2½[d]	27.18	25.00	47.82	0	4.17	2½[d]	33.72	28.90	33.72	3.65	4.0
					5.10						5.15						5.0
					6.06						6.12						6.0
5	34.04	23.40	41.49	1.07	5.05	5	25.80	23.74	45.44	5.02	5.00	5	31.46	26.96	31.46	10.10	5.0
					6.00						6.00						6.0
					6.93						7.00						7.0
7	32.54	22.37	39.67	5.42	6.84	7	24.48	22.52	43.09	9.91	5.00	7½	29.86	25.59	29.86	14.67	5.0
					5.75						6.00						6.0
					6.65						7.00						7.0
10	30.42	20.91	37.07	11.60	6.24	10	21.44	19.72	37.74	21.10	5.00	10	24.56	21.05	24.56	29.81	6.0
					7.07						6.00						7.0
					7.88						7.00						8.0
						(b) Loess Filler											
2½[e]	33.44	22.99	40.80	2.77	4.01	2½[e]	26.53	24.40	46.68	2.39	4.00	2½[e]	33.43	28.66	33.43	4.46	4.0
					4.97						5.00						5.0
					6.00						6.00						6.0
5	32.11	22.08	39.14	6.67	4.99	5	26.05	23.94	45.80	4.18	5.00	—	—	—	—	—	—
					6.00						6.00						—
					7.00						7.00						—
7	30.77	21.15	38.02	10.06	5.00	7	24.91	22.92	43.85	8.32	5.00	7	30.83	26.42	30.83	11.90	5.0
					6.00						6.00						6.0
					7.00						7.00						7.0
10	28.95	19.90	35.30	15.85	6.00	10	22.20	20.43	39.08	18.30	6.00	10	29.18	25.00	29.18	16.66	6.0
					7.00						7.00						7.0
					8.00						8.00						8.0

[a] By wt of total mix. [b] By wt of total aggregate. [c] Aggregates wash sieved to remove all -200 material.
[d] Aggregates dry sieved to remove all but 2.5 percent of -200 material.
[e] Filler added to bring total -200 content, including dust on aggregates to desired percentage.

TABLE 6

MIX PROPORTIONS FOR VARIOUS AGGREGATES AND FILLERS WITH ASBESTOS

Ocheydan Agg., Lime Filler							Garner Agg., Lime Filler							Garner Agg., Loess Filler						
-200[a] (%)	Aggregateb (%)				Asbestos[a] (%)	Asphalt[a] (%)	-200[a] (%)	Aggregate[b] (%)				Asbestos[a] (%)	Asphalt[a] (%)	-200[a] (%)	Aggregate[b] (%)				Asbestos[a] (%)	Asphalt[a] (%)
	3/4 in.	3/8 in.	Sand	Lime				3/4 in.	3/8 in.	Sand	Lime				3/4 in.	3/8 in.	Sand	Loess		
5	34.0	23.4	41.5	1.1	0	5.0	5	25.8	23.7	45.5	5.0	0	5.0	5	26.0	24.0	45.8	4.2	0	5.0
					0	6.0						0	6.0						0	6.0
					0	6.9						0	7.0						0	7.0
5	34.0	23.4	41.5	1.1	1	5.0	5	25.8	23.7	45.5	5.0	1	4.9	5	26.0	24.0	45.8	4.2	1	5.0
					1	5.9						1	5.8						1	6.0
					1	6.9						1	6.7						1	7.0
5	34.0	23.4	41.5	1.1	2	5.0	5	25.8	23.7	45.5	5.0	2	4.8	5	26.0	24.0	45.8	4.2	2	5.0
					2	5.9						2	5.7						2	6.0
					2	6.8						2	6.6						2	7.0
5	34.0	23.4	41.5	1.1	3	4.9	5	25.8	23.7	45.5	5.0	3	4.8	5	26.0	24.0	45.8	4.2	3	5.0
					3	5.8						3	5.7						3	6.0
					3	6.7						3	6.5						3	6.6
7	32.5	22.4	39.7	5.4	0	4.9	7	24.5	22.5	43.1	9.9	0	5.0	7	24.9	22.9	43.9	8.3	0	5.0
					0	5.8						0	6.0						0	6.0
					0	6.7						0	7.0						0	7.0
10	30.4	20.9	37.1	11.6	0	5.4	10	21.4	19.7	37.8	21.1	0	5.0	10	22.2	20.4	39.1	18.3	0	5.0
					0	6.2						0	5.9						0	6.0
					0	7.0						0	6.8						0	7.0
					0	7.9						—	—						0	8.0

[a] By st of total mix. [b] By wt of total aggregate.

total aggregate-filler. Mixes for three different asphalt contents in each minus 200-mesh filler group were prepared (Table 5). The asphalt contents were based on total mix.

A brief study concerning the effect of small quantities of asbestos on the properties of the mixes was included in the over-all study. Asbestos in 1, 2, and 3 percent amounts was added to the Ocheydan-lime, Garner-lime, and Garner-loess mixes containing 5 percent minus 200-mesh material (Table 6).

PREPARATION OF MIXES

All of the mixes used were prepared in the same manner to assure comparative results. Each of the several aggregates used was weighed in quantities sufficient to provide a 50-lb batch having the correct proportions (Tables 5, 6). After being weighed, the

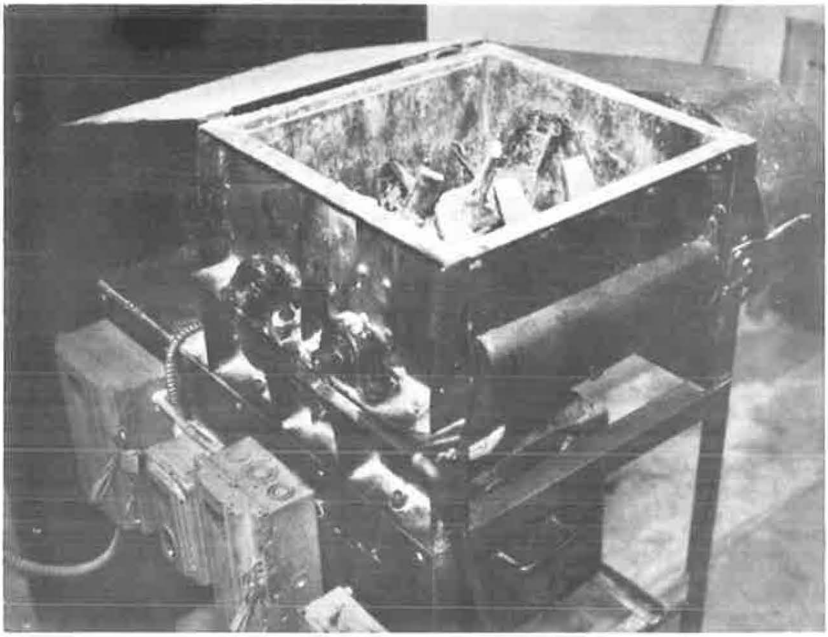


Figure 1. Laboratory twin shaft pugmill mixer.

aggregates were placed in pans with the coarse aggregate on the bottom. The fine aggregate and the filler were spread uniformly over the top. The loaded pans were covered and placed on a thermostatically controlled hot plate where they were heated overnight at 350 F to a uniform temperature of about 325 F.

After being heated, the proportioned aggregate and filler batch was placed in a preheated laboratory twin shaft pugmill mixer (Fig. 1). The aggregates were then dry mixed for 20 sec to assure uniform distribution of the ingredients, then the proper quantity of asphalt cement by weight at a temperature of 300 F was added and was mixed for 40 sec. The total mixing cycle was about 70 sec.

When mixing was completed, the asphaltic concrete was discharged into a pan which was immediately placed on a thermostatically controlled hot plate to keep the mix temperature between 250 and 275 F until the test specimens were molded.

Preparation of Test Specimens

Specimens were prepared for the Marshall stability, flow, percent voids, and percent voids filled with asphalt tests; for the Hveem stability, cohesiometer, both wet and dry, and percent voids in compacted specimen tests; and for the Iowa stability, both wet and dry tests. Additional specimens of each mix were prepared for extraction of asphalt and aggregate gradation checks and for the traffic simulator test.

The Marshall test specimens were prepared by standard procedure in a mechanical compactor (3). The specimens, compacted with 50 blows on each side, were 4 in. in diameter and approximately 2.5 in. high.

For the purpose of this investigation a modification was made in the standard Hveem procedure for compaction (3). The Hveem test specimens were compacted by the static load method rather than by the kneading compactor method for comparison with the Iowa State Highway Commission Stability Test which utilizes the static load method. Therefore, Hveem and Iowa stability and cohesiometer test specimens were compacted as follows: Approximately 1,200 gm of the mix at a temperature between 250 and 275 F were weighed out and placed in a 4-in. diameter mold preheated to 150 F. This mold is so constructed that a double plunger action is effected on the sample under load. Compaction of the specimen is in a static load compression machine capable of a uniform head speed. The specimen is loaded at a rate of 0.05 in./min until a load of 3,000 psi is reached. This load is maintained for 3 min and then released, thus completing the compaction of the specimen.

TESTING OF SPECIMENS

All specimens were allowed to cure in air at a temperature of about 75 F for 24 hr before testing.

Marshall Tests

The Marshall tests for stability, flow, percent voids in compacted specimen, and percent voids filled with asphalt were conducted in accordance with standard procedure (3). Before being tested for stability the specimens were submerged in a water bath at 140 F for 20 to 30 min.

Hveem Tests

The Hveem stability tests conformed to standard procedure (3). The test is outlined here for comparison with the Iowa stability test procedure.

Before being tested for Hveem stability the specimens are placed in an oven at 140 F for at least 1 hr. The specimen is then placed in the Hveem stabilometer, which in turn is placed in a compression test machine. The specimen is given an initial lateral pressure of 5 as indicated on the stabilometer; then the test load is applied at a rate of 0.05 in./min. Readings of the stabilometer gage are recorded at test loads of 500, 1,000, and at each subsequent 1,000 lb thereafter, up to a maximum of 6,000 lb. After attaining 6,000 lb, the load on the specimen is immediately reduced to 1,000 lb. The stabilometer gage is adjusted to 5 again. The dial gage on the pump is adjusted to zero, and the pump handle is turned at a rate of 2 turns per sec until a reading of 100 is obtained on the stabilometer gage. The number of turns required is recorded.

The Hveem stability value is then determined from

$$S = \frac{\dfrac{22.2}{(P_h)(D_2)}}{P_v - P_h} + 0.222 \tag{1}$$

in which

S = relative stability;
D_2 = displacement of specimen;
P_v = vertical pressure at 5,000 lb total load (400 psi); and
P_h = transmitted lateral pressure corresponding to P_v of 5,000 lb.

The computed value is recorded as Hveem stability-dry. A value of 35 or greater is deemed desirable for Iowa Type A mixes. The stabilometer gage reading at 5,000 lb total was recorded as Iowa stability-dry.

The Hveem cohesiometer tests were conducted according to standard procedure (3). The specimens were heated in an oven at 140 F for about 1 hr before testing. The cohesiometer value was determined by

$$C = \frac{L}{0.08H - 0.178H^2} \tag{2}$$

in which

C = cohesiometer value;
L = weight in gm causing failure; and
H = height of specimen in in.

In Iowa Type A mixes, a value for C of 50 or greater is considered desirable according to Hveem criteria. The cohesiometer values were recorded as cohesiometer-dry.

Iowa Tests

Preparation, compaction, and curing of test specimens for the Iowa highway tests followed the same procedure as that for the modified Hveem tests.

The test specimens are first immersed in a water bath at 140 F for at least 1 hr. The specimens are then tested by the procedure of the Iowa Highway Commission. The wet-heated specimen is placed in the Hveem stabilometer, which then is set in a compression test machine capable of maintaining a uniform rate of loading. An initial lateral pressure of 5, as indicated on the stabilometer gage, is placed on the specimen, followed by the test load at a rate of 0. 05 in. /min. Readings on the stabilometer gage are recorded at test loads of 500, 1, 000 and each subsequent 1, 000 lb thereafter, up to a maximum of 6, 000 lb. Up to this point the test procedure is the same as that of the standard Hveem.

The stabilometer gage reading at 5, 000 lb vertical load, 400 psi, is the Iowa stability value. A value of 60 or less is deemed desirable for Iowa Type A mixes. These results are recorded at Iowa stability-wet. These tests were extended to acquire data necessary to calculate Hveem stability values, recorded as Hveem stability-wet.

Cohesiometer. —Cohesiometer tests were also conducted on the Iowa specimens in the same manner as for the Hveem specimens except that these specimens were heated in a water bath at 140 F rather than in an oven. These cohesiometer values are listed as cohesiometer-wet.

Extraction Tests

As a check on mix proportions, a representative sample of each mix was extracted in a rotarex extractor in accordance with standard ASTM rotarex procedure (ASTM Designation 1097) (4) with CCl_4 as the solvent. These tests confirmed that all mixes were in compliance with their respective designated proportions.

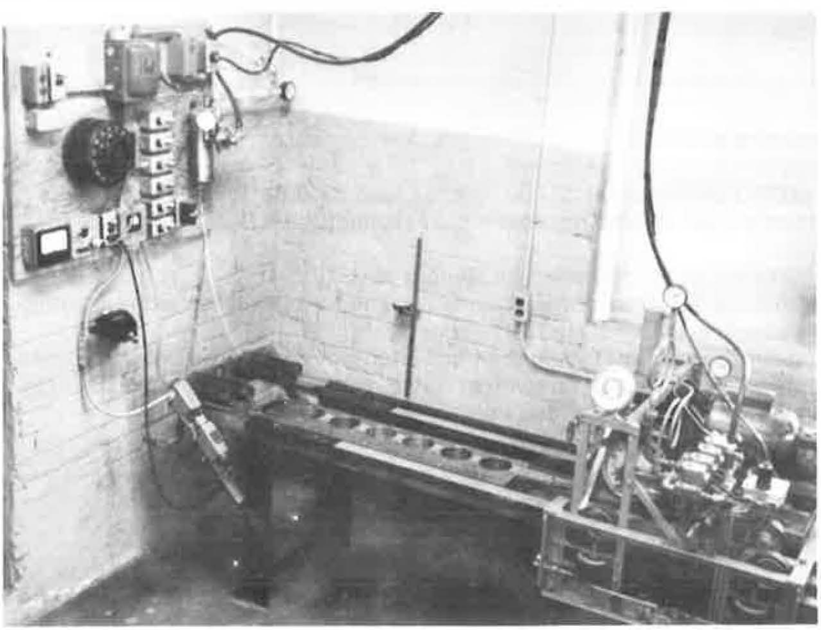


Figure 2. Iowa State University traffic simulator.

Traffic Simulator Tests

Trafficability tests were conducted on a compacted specimen of each mix by a traffic simulator designed and built by the Bituminous Research Laboratory of Iowa State University (Fig. 2). In the traffic simulator, Marshall and Hveem compacted test specimens, heated to and maintained at a temperature of 100 F in the machine, are subjected to a forward driving wheel load equivalent to an 80-psi tire pressure. The drive wheel, 8 in. in diameter and $1\frac{1}{2}$ in. wide, fitted with a soft, solid rubber tire travels over the specimen surface at about 4 mph. Measurements on the surface of the specimen along the wheel track are taken initially and after each 1, 000 passes of the wheel up to 5, 000 wheel passes. Displacement of the surface of the specimen is then calculated and recorded in 0. 001 in. at 1, 000, 3, 000, and 5, 000 passes.

ANALYSIS OF TEST RESULTS

A number of variables, such as aggregates, fillers, test procedures, and the filler and asphalt contents in the mixes, were involved in the investigation. The effect of any one of the variables on the properties of the mixes may be analyzed and evaluated and compared with the effect of any other variable.

The analysis of results in this discussion, however, is limited to the effect of the fillers on the properties of the mix, because this was the primary purpose of the study. Therefore, an analysis is presented of the effect of fillers on the stability, void content, cohesion, and trafficability of the mixes. Each value given is the result of averaging the test results obtained from three Marshall compacted specimens, and from five static load Hveem and Iowa highway compacted specimens.

Effect of Fillers on Stability of Mixes

Marshall Stability—Dynamic Compaction. —In mixes containing 5 percent asphalt and lime filler, stability dropped sharply at 5 percent filler content for both the Ocheydan gravel and Garner hard limestone aggregates (Fig. 3). The loss in stability at this point may be attributed either to the relationship between filler and asphalt or to the orientation of particles or to both. The stability increased with increase in lime filler content above

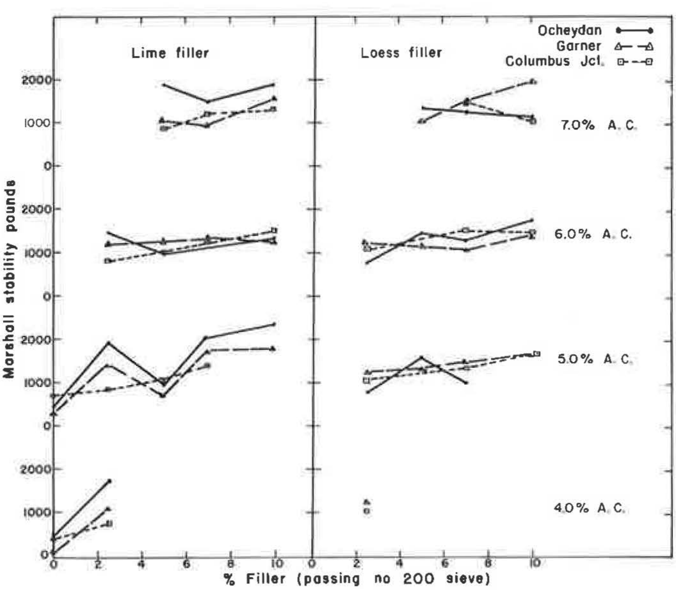


Figure 3.

5 percent. An entirely different trend was shown in mixes containing 5 percent asphalt with pulverized loess as a filler. Ocheydan mixes attained their maximum stability at 5 percent filler content, and Garner and Columbus Junction mixes increased in stability with increase in filler content (Fig. 3). Loess filler appeared to have a smaller effect on stability than lime fillers over a range of filler content. This might be attributed to the interrelationship of the type of filler and the asphalt.

Mixes with lime as a filler and with 6 percent asphalt varied less in stability than those with 5 percent asphalt. The Ocheydan mixes dropped in stability at 5 percent filler content. The Garner mixes attained their maximum stability at 7 percent filler content, and the Columbus Junction mixes showed a steady increase in stability with increase in filler content. The effect of the lime filler with regard to type of aggregate was at a minimum at 7 percent filler content, as indicated by a 250-lb difference only in stability for all aggregate mixes (Fig. 3). When pulverized loess was used as a filler, Ocheydan and Garner mixes showed a slight loss of stability at 7 percent filler. The Columbus Junction mixes attained their maximum stability at this point. The change in stability over the range of loess filler content was somewhat wider than that for lime filler. The stabilities of the loess mixes are slightly higher than those of the lime filler mixes (Fig. 3).

Both the Ocheydan and Garner mixes with 7 percent asphalt had their lowest stability at 7 percent lime filler content. The Columbus Junction mixes increased in stability with increase in lime filler content. But mixes with pulverized loess as the filler gave different results. The Ocheydan mixes showed a slight loss in stability with increase of filler above 5 percent. The Garner mixes showed a sharp increase in stability above 5 percent filler content. And the Columbus Junction mixes dropped sharply in stability above 7 percent filler content (Fig. 3).

A general statement about the effect of the quantity of filler in a mix on its Marshall stability cannot be made. Character of aggregate, type of filler, and asphalt content all must be considered.

Some conclusions from the results of the Marshall stability test procedure are:

1. The quantity of lime filler in a mix had a minimal effect on Marshall stability when the mix contained 6 percent asphalt, regardless of the type of aggregate.
2. The quantity of pulverized loess in a mix had a less effect on Marshall stability than the quantity of lime filler regardless of asphalt content and type of aggregate. This was probably due to the closer interrelationship of the loess filler and the asphalt.
3. The Marshall stability of mixes with soft limestone aggregate increased with increase in filler content regardless of type of filler or asphalt content. This may have been due to a higher degree of degradation from dynamic compaction in this type of aggregate than in the harder aggregates which gave denser specimens for equivalent asphalt and filler contents.
4. Mixes containing loess filler are not as sensitive to increase in asphalt contents as are those with lime fillers. This may be because the loess absorbed or became suspended in the asphalt. Additional studies being conducted appear to favor the latter.

Hveem Stability—Static Compaction. —The Ocheydan aggregate mixes with lime as a filler and with 5 percent asphalt increased in Hveem stability materially with increasing filler content (Fig. 4), perhaps because the increased quantity of filler generates greater internal friction in the mix in relation with the gravelly character of the aggregate. The Garner mixes attained their maximum stability at 5 percent filler, then dropped sharply in stability at 7 percent filler, only to recover again sharply at 10 percent filler content. This behavior may be attributed to the asphalt-filler relationship acting on the internal friction of the mix. There seems to be no direct relationship between this behavior and the void content and the cohesion of these mixes. In the Columbus Junction mixes the quantity of filler between $2\frac{1}{2}$ and 7 percent seemed to have little or no effect on the stability. In mixes with loess as a filler, the Ocheydan mixes increased in stability with increase in filler content much the same as those containing lime as a filler, and for probably the same reason. The Garner mix attained its peak stability at 10 percent filler with considerably lower stabilities at 5 and 7 percent filler content.

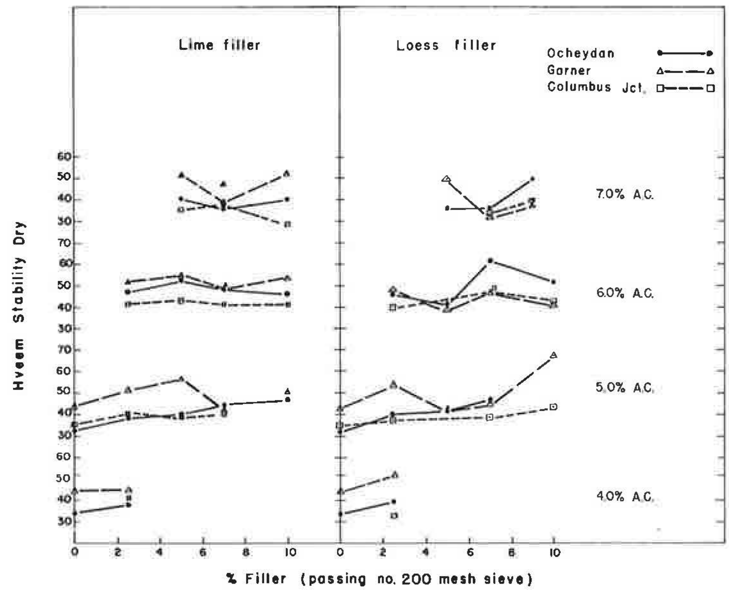


Figure 4.

This behavior does not correlate with void content but does to some degree with cohesion. The highest cohesion is attained at 10 percent filler content. Possibly the intimate relationship between loess filler and the asphalt may be responsible for this behavior. In the Columbus Junction mixes the quantity of filler in the mix seemed to have no effect on stability.

At 6 percent asphalt, Ocheydan mixes with lime filler attained their peak stability at 5 percent filler and then decreased in stability with increase in filler content. The Garner mixes also attained their maximum stability at 5 percent filler, then dropped at 7 percent and recovered stability at 10 percent filler content. The Columbus Junction mixes also reached a maximum stability at 5 percent filler content and then fell off in stability with increased filler; however, the effect of filler content on stability was relatively small. At 6 percent asphalt content the stabilities of all mixes with lime filler were generally higher than similar mixes containing 5 percent asphalt, and the effect of the filler was not quite so marked. When loess was used as a filler, all mixes attained their maximum stability at 7 percent loess filler content, regardless of aggregate type. In the Ocheydan mixes it appears that an interrelation between filler and asphalt occurs which yields peak stability at a sacrifice of cohesion. The Garner and Columbus Junction mixes attained peak stability with high cohesion at 7 percent filler. The shift of peak stability from 5 percent lime filler to 7 percent loess filler might have been caused by the finer gradation of the loess and a more intimate relation between the loess and the asphalt.

The Ocheydan and Garner mixes with lime as a filler and with 7 percent asphalt had their lowest stability at 7 percent filler, with relatively high stabilities at 5 and 10 percent filler. The Columbus Junction mixes, however, showed peak stability at 7 percent filler. All mixes, regardless of type of aggregate, had about the same stability at 7 percent filler content. This group of mixes clearly exhibits the effect of the relationship of filler and asphalt content in a mix. With loess as filler, all mixes behaved much the same with lowest stability at 7 percent filler content. Here also, as in the lime filler mixes, the mixes with loess as a filler have nearly the same stability.

As with Marshall stability, a general statement about the effect of the quantity or type of filler in a mix on its Hveem stability cannot be made. However, the effect of the filler on the properties of mixes differed materially when measured by the Marshall and the Hveem methods of determining stability. This is no doubt due to the differing methods of compaction of specimens and methods of test.

Some conclusions from the results of the Hveem stability test procedure are:

1. Most mixes with lime filler attained their highest stability at 5 percent filler content and those with loess, at 7 percent filler content.
2. The quantity of filler contained in a mix had the least effect on the mixes with 6 percent asphalt, regardless of aggregate type. This was also noted in the Marshall test.
3. All mixes generally, regardless of filler content, met minimum Hveem stability criteria.
4. Loess fillers gave excellent stability values and lime fillers showed better results at lower filler contents, but in the higher percentages loess gave the better stability values (4).

Iowa Stability—Static Compaction. —A value of 60 psi is used as a maximum for acceptable mixes in the Iowa stability—static compaction test. Consequently, the lower the test value the better the stability of the mix (Fig. 5).

In mixes containing 5 percent asphalt and lime as a filler, the stabilities of the Ocheydan and Garner mixes fluctuated slightly but tended toward increasing stability with increasing filler content. The stabilities were generally greater with the peak at 5 percent filler, however, the effect of filler content on stability was small. The stability of the Columbus Junction mixes was considerably lower than that of the other mixes and the quantity of filler had practically no effect. When loess was used as a filler, the stability of the Ocheydan mixes improved with increase of filler from $2\frac{1}{2}$ to 7 percent. That of the Garner mixes fluctuated markedly and showed a sharp loss at 5 percent, an equally sharp gain at 7 percent, and another sharp loss at 10 percent of filler content. This behavior of the Garner mixes seems to correlate with the void content of the test specimens. Because the specimens were tested for stability after immersion in hot water, absorption of moisture by the specimen and its effect on the loess filler may be responsible for this behavior. The quantity of filler up to 7 percent had little effect on the stability of the Columbus Junction mixes; however, at 10 percent filler content stability showed a decided improvement.

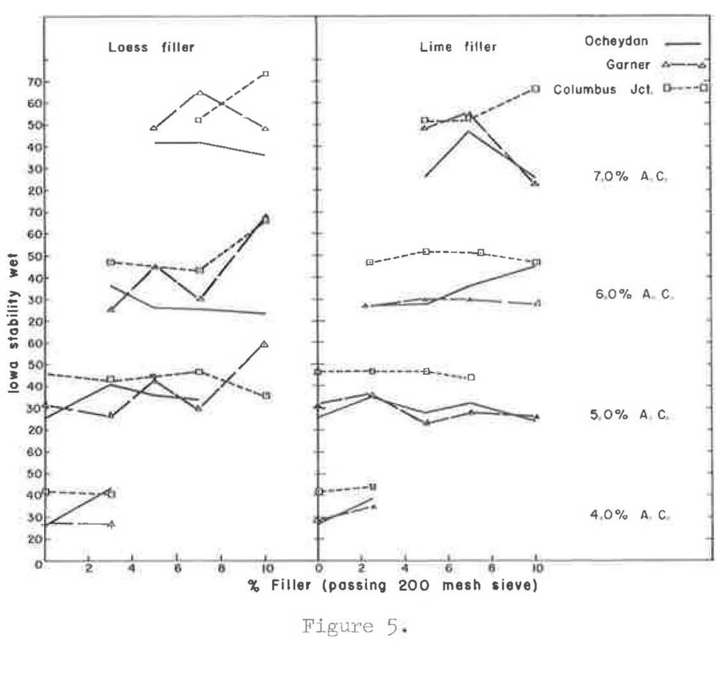


Figure 5.

In mixes with 6 percent asphalt and lime as a filler the quantity of filler in a mix had no effect on the stability of the Garner and Columbus Junction mixes. The loss in stability was sharp, however, with filler contents in excess of 5 percent in the Ocheydan mixes. When loess was used as a filler, the Ocheydan mixes improved in stability somewhat with filler contents in excess of 5 percent. The Garner mixes fluctuated widely in stability with increase in filler; stability dropped at 5 percent, recovered at 7 percent, and dropped to an unacceptable level at 10 percent filler contents. The Columbus Junction mixes were little affected by filler contents up to 7 percent, and then dropped sharply to an unacceptable level at 10 percent. The behavior noted in this group of mixes seems to be related almost directly to the voids in the test specimens.

In the 7 percent asphalt and lime filler mixes, the Ocheydan and Garner mixes lost stability from 5 to 7 percent regained stability sharply at 10 percent filler content. The Columbus Junction mixes had relatively low stability at 5 and 7 percent filler contents and dropped sharply to unacceptable values with 10 percent lime filler. There is no correlation with voids in test specimens. With loess as a filler, the Ocheydan mixes improved slightly in stability with increase in filler content; the Garner mixes lost stability at 5 to 7 percent and recovered it at 10 percent filler content. The Columbus Junction mixes lost stability from 7 to 10 percent filler content and dropped to unacceptable values at 10 percent loess filler content.

Effect of Filler Content on Stability in General. —The test results indicate that no general statement concerning the effect of fillers on the stability of an asphaltic concrete mix can be made. The effect of the filler on stability of a mix varies with the method of test used to determine stability, the character of the aggregates, and the quantity of asphalt contained in the mix.

The effect of the filler in mixes containing the same aggregate and same quantity of asphalt on stability as measured by the different methods was inconsistent and frequently showed a direct reversal of stability between methods. A test method often was quite sensitive to the variation in filler content with some mixes and was least sensitive with others.

The effect of fillers also varied with asphalt content of a mix. The amount of variation depended on the method used to determine stability. The least variation noted, regardless of method of test, was in mixes containing 6 percent asphalt. In such mixes the effect of the filler on stability seemed to be minimal.

The type of aggregate contained in a mix also had a definite bearing on the effect of a filler on stability. The effect of a filler was much greater in mixes containing hard aggregates, such as the Ocheydan and Garner. The filler usually had little effect with a softer aggregate, such as the Columbus Junction.

Effect of Fillers on Void Contents

Dynamic Compaction—Marshall Method. —Void contents of the test specimens (Fig. 6) compacted by the Marshall method of dynamic compaction were determined by standard procedures (5). The results were analyzed on the basis of type and quantity of filler, type of aggregate, and quantity of asphalt contained in the mix. The results indicated that the voids in the compacted specimens decreased with increased quantity of asphalt, regardless of type or quantity of filler or type of aggregate.

In mixes using lime as the filler a rather unusual effect was observed. The Ocheydan mixes showed an increase in voids at 5 percent filler and a decrease at 7 and 10 percent filler content in mixes containing 5 and 6 percent asphalt. At 7 percent asphalt the lowest void content appeared at 7 percent filler with an increase in voids at 10 percent filler content. In the Garner mixes with 5 percent asphalt, the voids increased similarly at 5 percent filler and decreased at 7 percent, and again increased at 10 percent filler content. At 6 and 7 percent asphalt contents the lowest voids were at 5 percent filler, followed by a relatively sharp increase in voids with increased filler content. The Columbus Junction mixes with 5 percent asphalt increased in voids up to 5 percent filler, then decreased with additional filler content. At 6 and 7 percent asphalt content the voids decreased with filler up to 7 percent and then increased at 10 percent filler.

When loess was used as a filler, the Ocheydan mixes attained their lowest void content at 5 percent filler and then increased sharply with additional filler, regardless of

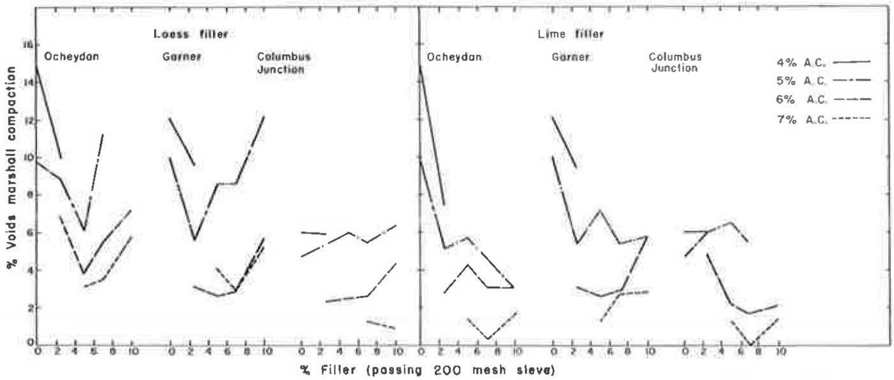


Figure 6.

asphalt content. The Garner mixes with 5 percent asphalt increased in voids with increase in filler content. At 6 percent asphalt the lowest void content was at 5 percent filler and at 7 percent asphalt, at 7 percent filler content. In the Columbus Junction mixes with 6 and 7 percent asphalt, voids increased with increase in filler content.

Static Compaction.—The voids in specimens compacted by static means (Fig. 7) were determined in the same manner as for those compacted by dynamic means (5).

With lime as a filler, the specimens of Ocheydan mixes compacted by the static method behaved much the same as those compacted by dynamic means. The rise in voids at 5 percent filler was followed by a decrease with increase in filler. The behavior of Garner and Columbus Junction mixes varied only slightly. The Garner mixes attained their lowest voids at 5 percent filler; the Columbus Junction mixes attained theirs at 7 percent. The method of compaction had slight effect on these mixes, because much the same trends in void contents with filler content were noted.

When loess was the filler in the Ocheydan mixes with 5 percent asphalt, the lowest void content was at 5 percent filler. At 6 percent asphalt the greatest void content was at 5 percent filler, followed by a decrease in voids with increase in filler content. In the Garner mixes the voids increased rather sharply with increase in filler content. In the Columbus Junction mixes with 5 percent asphalt the voids increased up to 7 percent filler, then decreased at 10 percent filler content. At 6 percent asphalt, voids increased with the increase in amount of filler.

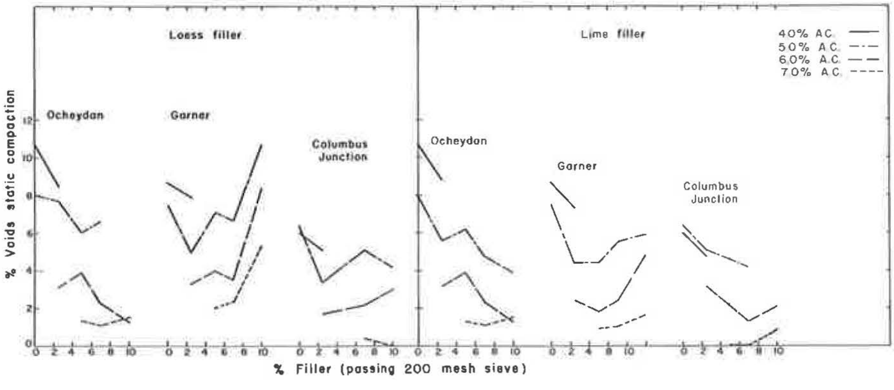


Figure 7.

General Effect of Fillers on Voids. —No specific trends were shown. The void content of a compacted specimen, however, was affected by its asphalt content to a greater degree than by either the type or quantity of filler. Often the method of compacting specimens had only slight effect on void content. Mixes with loess as a filler usually had higher void contents than similar lime mixes. This may be due to the method of determining voids or to a closer relation between filler and asphalt in which more of the loess, because of its finer particle size or mineral properties, becomes suspended in the asphalt.

Effect of Fillers on Cohesion

Only the statically compacted specimens were tested by the Hveem cohesiometer. In the test, normal procedures were followed for specimens heated in an oven at 140 F to determine dry cohesion. Other specimens were immersed in a water bath at 140 F to determine wet cohesion (Fig. 8). According to Hveem cohesiometer criteria, cohesion values above 50 are desirable for high-type asphaltic concrete mixes.

When lime was the filler, the Ocheydan mixes had their lowest cohesion at 5 percent filler which then increased sharply with increase in filler content. The Garner mixes with 5 percent filler also were lowest in cohesion, which increased gradually with additional filler except for the mix containing 7 percent asphalt which increased rather sharply. The Columbus Junction mixes at 5 and 6 percent asphalt increased in cohesion with increase in filler up to 7 percent filler content and then decreased with the further increase of filler; but mixes with 7 percent asphalt had their lowest cohesion at 7 percent filler. Regardless of the variations in cohesion, all mixes had cohesion values well above the 50 minimum criteria. No correlation between cohesion, stability, and voids was found. The lime filler seemed to exert its greatest effect on the Ocheydan mixes and the least on the Garner mixes.

When loess was the filler, Ocheydan mixes with 5 percent asphalt attained their highest cohesion with 5 percent filler, and then decreased in cohesion with additional filler. At 6 percent asphalt the cohesion increased with increase in filler content. The Garner mixes, regardless of asphalt content, increased in cohesion sharply from 5 to 7 percent filler and then increased gradually in cohesion with additions of filler up to 10 percent. The Columbus Junction mixes with 6 percent asphalt increased in cohesion with increase in filler, and those with 5 percent asphalt developed an unusually high cohesion value of 920 with 7 percent filler. Loess as a filler did not have as much effect on the cohesion of the Ocheydan and Garner mixes as did the lime, however, its effect on the Columbus Junction mixes was extraordinary. The cohesion values of all mixes with loess were above the minimum.

Filler itself generally has no effect on cohesion. However, there is a define relationship between filler and asphalt that has a decided effect on the cohesion of a mix.

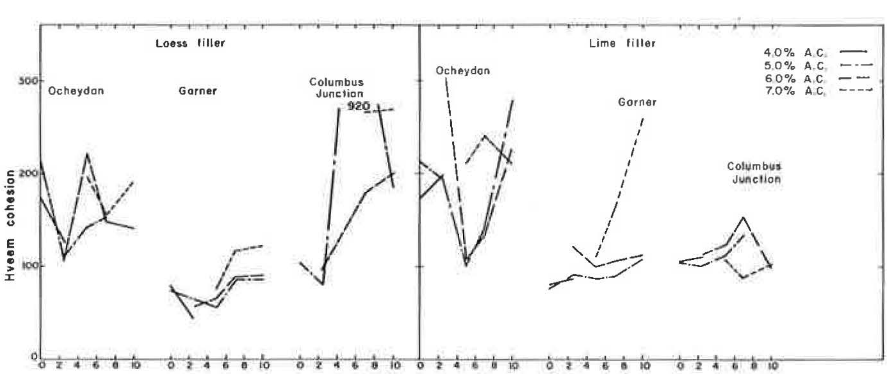


Figure 8.

The filler combined with asphalt evidently modifies the physical properties of the asphalt which then acts as the cementing agent to make it stronger, tougher, and more adhesive. A separate study has been undertaken to explore the extent of the relationship between filler and asphalt to seek an explanation of its effect not only on cohesion but also on voids and stability. Wet cohesion values were generally higher than dry cohesion values for most mixes.

Effect of Filler on Trafficability

Compacted specimens of all mixes were tested in the traffic simulator to compare their resistance to displacement under moving loads (Fig. 9). In the traffic simulator tests, the displacement in excess of 0.125 or 1/8 in. of a mix, after 5,000 passes by the wheel load, seems to indicate that the mix may not resist rutting or channelizing under heavy traffic volumes.

In mixes with lime as filler, the dynamically compacted specimens of Ocheydan and Garner mixes with 5 percent asphalt improve in trafficability with increase in filler content, attaining the arbitrary criteria of maximum displacement of 1/8 in. for 500 passes with filler contents of 5 percent and greater. In similar mixes statically compacted, the behavior is much the same as noted for mixes dynamically compacted, except that a slight loss in resistance to traffic occurs with 10 percent filler content. The Columbus Junction mixes all showed excellent trafficability, well within the criteria, regardless of filler content or method of compaction. At 6 percent asphalt the dynamically compacted mixes varied widely in behavior. The Ocheydan mixes improved in trafficability with increase in filler content up to 7 percent and met criteria at 5 percent. At greater than 7 percent filler the mixes went above the limit. The Garner mixes failed to meet criteria in mixes containing less than 10 percent filler. All Columbus Junction mixes met trafficability criteria. In statically compacted mixes all those up to 7 percent filler met the criteria. At greater than 7 percent filler, the Ocheydan and Garner mixes tended to show distress under traffic. At 7 percent asphalt all Ocheydan and Garner mixes failed to meet criteria, regardless of filler content or method of compaction, although these mixes met stability requirements for high-type pavements. All Columbus Junction mixes met trafficability criteria.

The specimens from dynamically compacted Ocheydan and Columbus Junction mixes with loess as a filler and with 5 percent asphalt are within trafficability criteria with

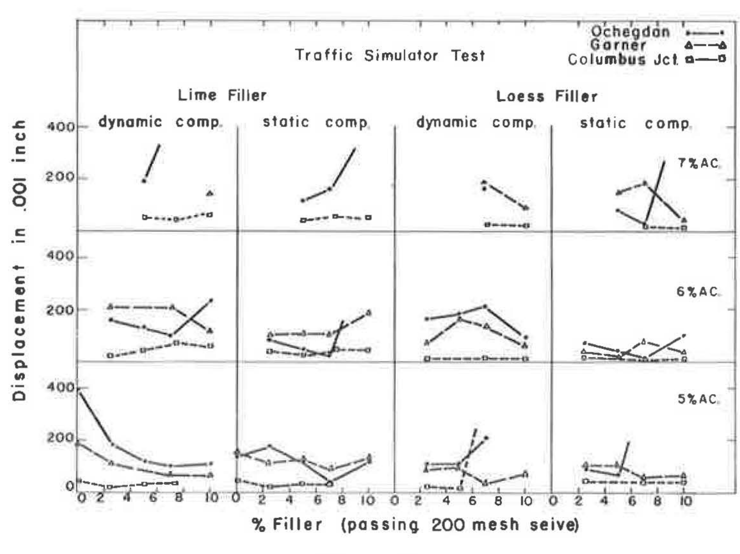


Figure 9.

filler contents up to 5 percent, but above that they lose trafficability sharply. The Garner mixes met criteria, with best results at 7 percent filler content. In mixes statically compacted, both the Garner and Columbus Junction mixes met criteria over the entire range of filler content. The Ocheydan mixes with more than 5 percent loess filler failed. At 6 percent asphalt, dynamically compacted, only the Ocheydan mix with 10 percent filler met criteria. The Garner mixes with 7 percent filler and more, and the Columbus Junction mixes over the full range of filler content, met criteria. All statically compacted mixes met criteria over the entire range of filler content. At 7 percent asphalt, all dynamically compacted Ocheydan mixes failed to meet criteria. The Garner mix with 10 percent filler and all Columbus Junction mixes with 7 percent or more of filler met criteria. The same statically compacted Ocheydan mixes with 5 and 7 percent filler met the criteria, as did the statically compacted Garner mixes with 5 and 10 percent filler and the statically compacted Columbus Junction mixes with more than 7 percent filler. The statically compacted mixes with loess showed a better resistance to traffic than similiar mixes with lime over the range of asphalt content.

No direct relationship between either type of quantity of filler and trafficability is evident from these test results. The degree of resistance to traffic seemed to depend largely on asphalt content and method of compaction. Generally, lower asphalt contents and static compaction yielded the higher resistance to displacement in the harder aggregate mixes. Surprisingly, the softer Columbus Junction aggregate mixes displayed the best resistance to traffic displacement, regardless of asphalt or filler content and method of compaction. No direct correlation of trafficability with void content or with stability of mixes is evident. Many mixes with high stability failed to meet trafficability criteria. Several mixes with comparatively low stability showed excellent resistance to displacement under a moving wheel load. This confirms field observations that mixes of apparently adequate stability show considerable distress by rutting or channeling in the wheel tracks of pavements carrying heavy traffic.

Effect of Asbestos as Additive

A special study was undertaken to ascertain some of the effects on mix properties of the addition of small quantities of asbestos (6). The mixes used in this phase of the investigation included the Ocheydan aggregate with 5 percent lime as the filler, the Garner aggregate with 5 percent lime and with 5 percent loess as the filler and with 5, 6, and 7 percent asphalt contents. Other mixes with 7 and 10 percent filler were used only for comparison (Table 6). Test mixes were prepared with the basic proportions and 1, 2, and 3 percent asbestos by weight with 5, 6, and 7 percent asphalt content. The mixes were tested for Marshall, Hveem, and Iowa stability, voids in compacted specimens, Hveem cohesion, and trafficability in the traffic simulator in the manner described previously (Figs. 10, 11, 12, and 13).

Effect of Asbestos on Stability. —The Marshall stabilities of the Garner-lime mixes were materially improved by the addition of asbestos, with 3 percent addition making the greatest improvement. In the Garner-loess mixes no improvement in Marshall stability was secured with the addition of 1 or 2 percent asbestos, but stability increased greatly when 3 percent asbestos was added. The addition of asbestos increased the stabilities of the Ocheydan-lime mixes containing 5 and 6 percent asphalt, but markedly reduced the stability of mixes with 7 percent asphalt.

The results of the test indicate that addition of asbestos lowered the Hveem stabilities of the Garner-lime mixes. The Hveem stabilities of the Garner-loess and the Ocheydan-lime mixes were materially improved by the addition of asbestos, and the 2 percent addition provided the greatest increase.

The Iowa stability of the Garner-lime mixes was adversely affected by the addition of 2 to 3 percent asbestos in mixes containing 6 percent or less of asphalt, but was materially improved by addition to mixes containing 7 percent asphalt. The addition of asbestos to the Garner-loess mixes generally improved their stabilities. The Iowa stabilities of the Ocheydan-lime mixes were adversely affected by the addition of 1 percent asbestos, and addition of greater amounts had little or no beneficial effects.

The test results clearly indicate that the method of determining the stability of a mix had a decided bearing on the evaluation of the effect of the quantity of asbestos added to a mix.

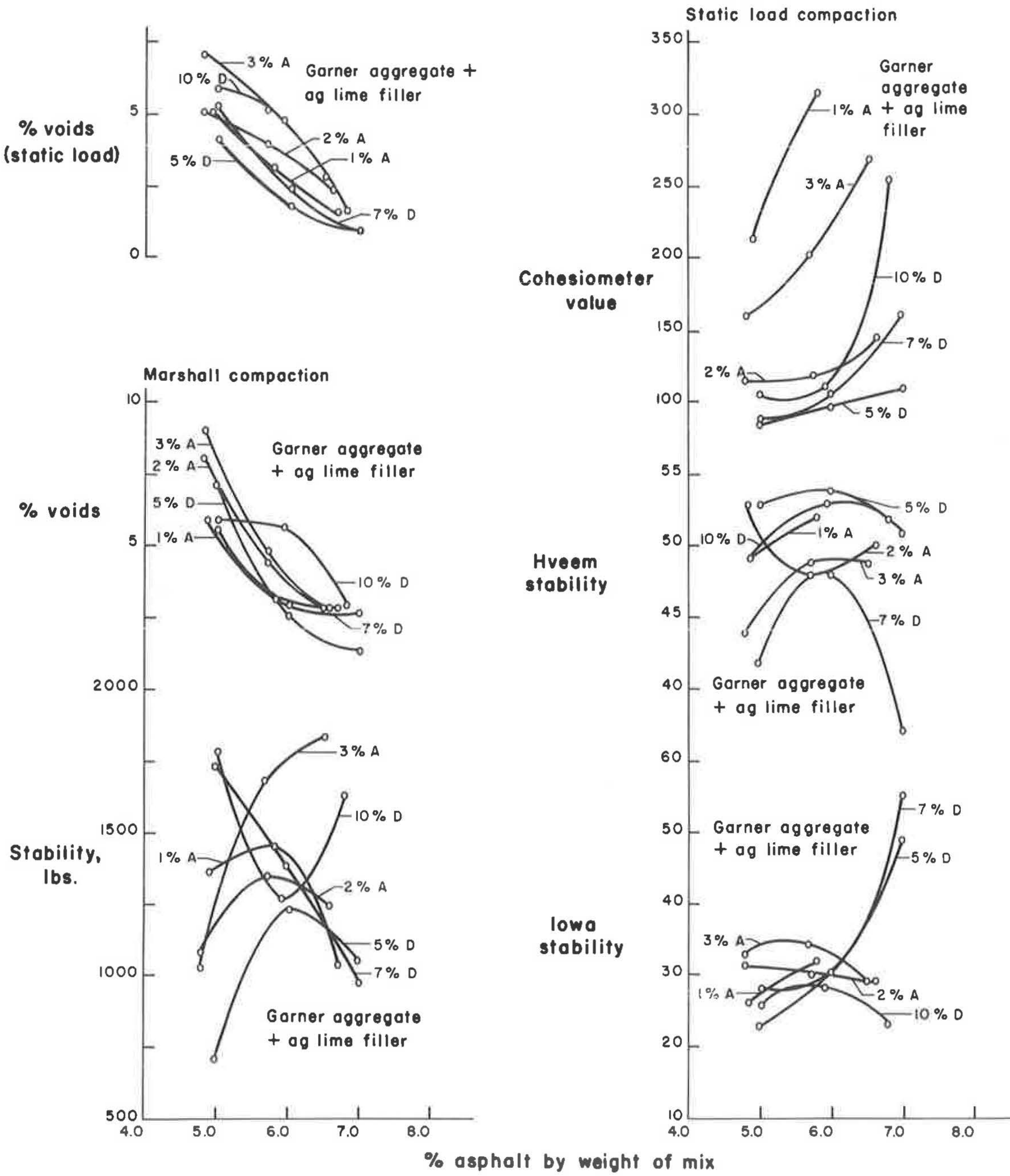

% voids (static load)
Garner aggregate + ag lime filler
3 % A
10 % D
2 % A
1 % A
5 % D
7 % D
Marshall compaction
% voids
Garner aggregate + ag lime filler
10 % D
7 % D
Stability, lbs.
3 % A
10 % D
1 % A
2 % A
5 % D
7 % D
Garner aggregate + ag lime filler
Static load compaction
Cohesiometer value
Garner aggregate + ag lime filler
1 % A
3 % A
10 % D
7 % D
2 % A
5 % D
Hveem stability
5 % D
10 % D
1 % A
2 % A
3 % A
7 % D
Garner aggregate + ag lime filler
Iowa stability
7 % D
Garner aggregate + ag lime filler
5 % D
3 % A
1 % A
2 % A
10 % D
% asphalt by weight of mix

Figure 10.

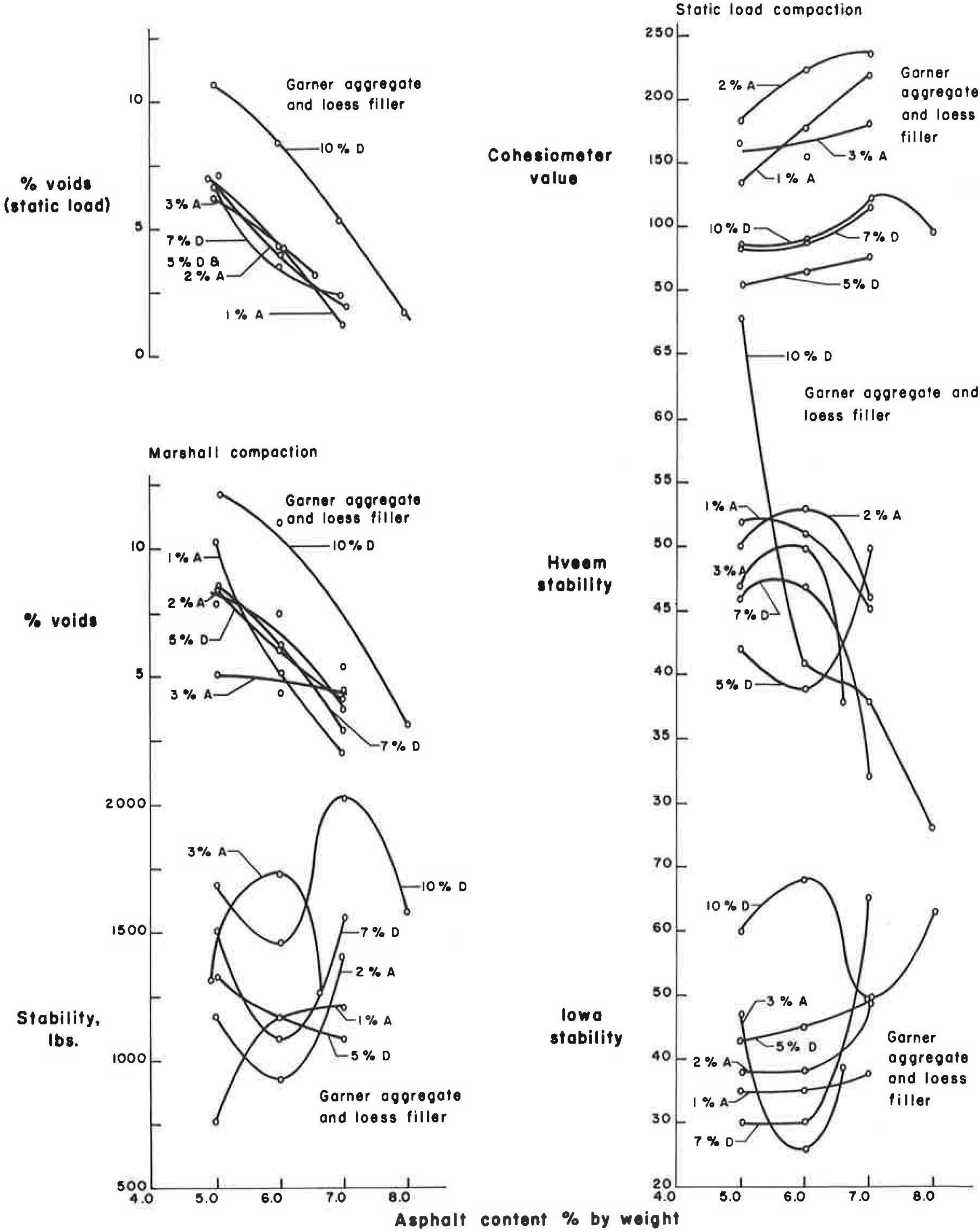
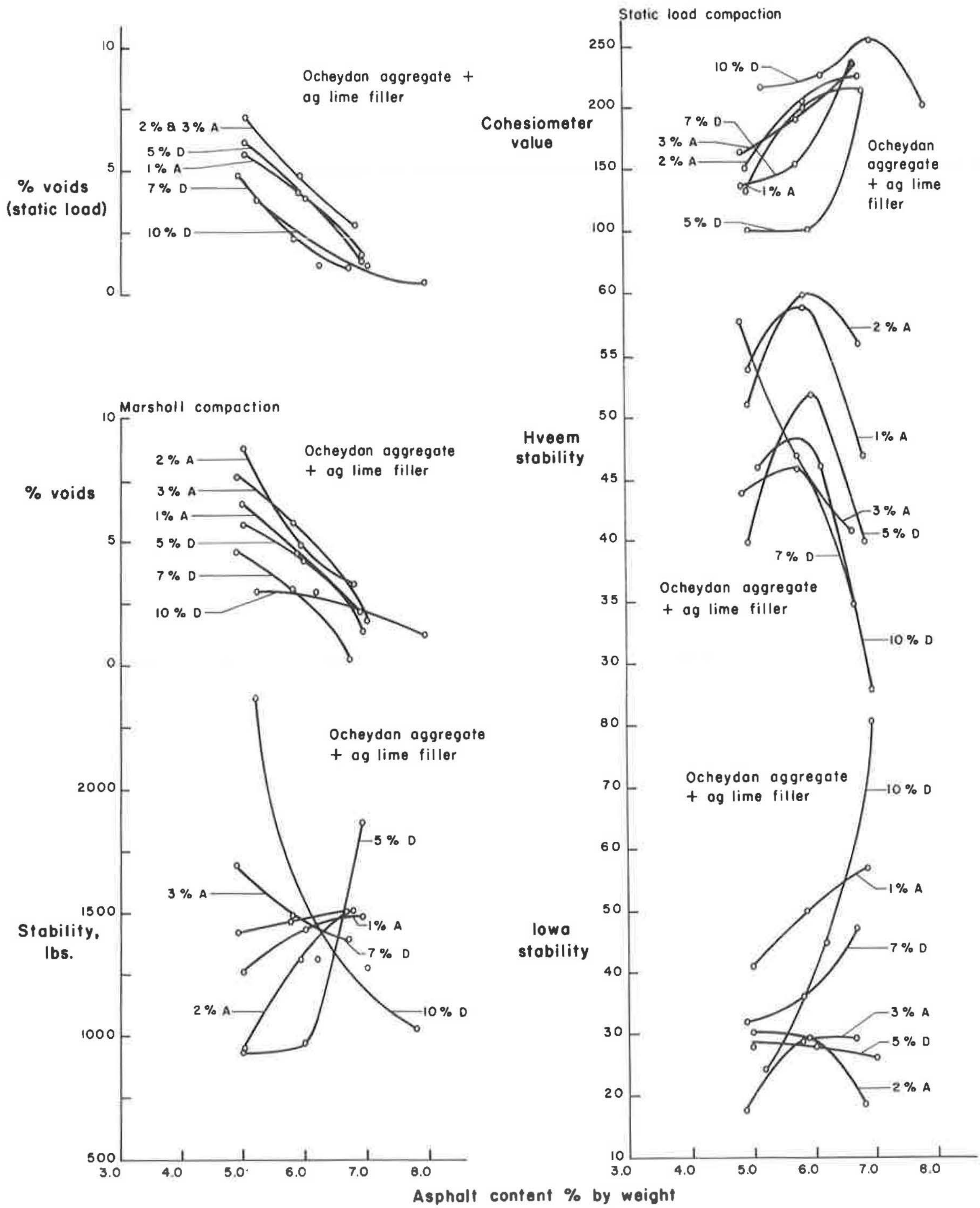

% voids (static load)
Garner aggregate and loess filler
10 % D
3 % A
7 % D
5 % D & 2 % A
1 % A
Marshall compaction
% voids
Stability, lbs.
2 % A
3 % A
5 % D
7 % D
1 % A
Static load compaction
Cohesiometer value
Hveem stability
Iowa stability
Asphalt content % by weight

Figure 11.



Static load compaction
Ocheydan aggregate + ag lime filler
2 % & 3 % A
5 % D
1 % A
7 % D
10 % D
% voids (static load)
Cohesiometer value
3 % A
2 % A
5 % D
Marshall compaction
% voids
Hveem stability
Stability, lbs.
Iowa stability
Asphalt content % by weight

Figure 12.

Effect of Asbestos on Voids. — In the Garner-lime mixes, dynamically compacted, addition of 1 percent asbestos reduced voids in mixes having 6 percent or less of asphalt, but 2 and 3 percent asbestos increased voids, regardless of asphalt content. In the same statically compacted mixes, voids increased with addition of asbestos.

In the dynamically compacted specimens of Garner-loess mixes, the addition of 1 percent asbestos reduced voids in the mixes with 5 percent or more asphalt, but 2 percent asbestos had no effect. The addition of 3 percent asbestos reduced voids in mixes having 6 percent or less asphalt. The statically compacted specimens of the same mixes showed little change in voids with added asbestos.

The addition of asbestos to the dynamically compacted specimens from Ocheydan-lime mixes increased voids. Those statically compacted showed very little change in voids.

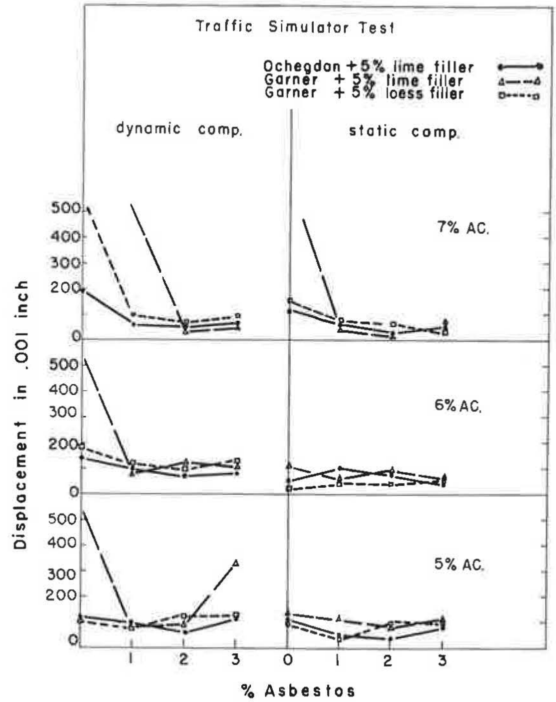


Figure 13.

In general, the addition of asbestos had only minor effect on the void content of a mix, regardless of the method of compaction or type of materials in the mix.

Effect of Asbestos on Cohesion. —In all mixes tested the addition of asbestos produced a material increase in the cohesion of the mix. The addition of 1 percent asbestos in the Garner-lime mixes and the addition of 2 percent asbestos in the Garner-loess and Ocheydan-lime mixes yielded the greatest improvements.

Effect of Asbestos on Trafficability. —In general, the addition of asbestos improved the trafficability of all mixes tested. The extent of the improvement varied with the method of compaction and with the asphalt content of the mix (Fig. 13). The addition of 2 percent asbestos appeared to give the best results with most mixes tested and the greatest improvement in mixes with 7 percent asphalt.

CONCLUSIONS

From the evaluation and analysis of test results obtained in the course of this investigation, the following conclusions were drawn:

1. A general statement concerning the effect of fillers on the several properties of an asphaltic concrete, such as stability, voids in compacted specimens, cohesion, and trafficability, cannot be made. However, the specific effect of a filler on mixes having the same aggregates and asphalt content can be determined.
2. The effect of fillers on the stability of a mix varies with the method used to determine its stability, the asphalt content, and the characteristics of the aggregates used in the mix.
3. Different methods of determining stability indicate different patterns of the effect of fillers on the stability of a mix, because the methods differ in their sensitivity to the quantity of filler present in a mix.
4. Fillers have a more marked effect on the stability of mixes containing hard aggregates than on those containing softer aggregates.

5. There is a definite intimate relationship between filler and asphalt in a mix. This relationship has a marked effect on stability and cohesion. The asphalt-filler combination appears to modify the properties of the asphalt acting as the cementing agent of the aggregates, making it apparently stronger, tougher, and more adhesive at optimum conditions. The relationship between filler and asphalt, particularly as to form and degree prevalent in mixes, should be studied further.

6. The quantity of filler has the least effect on stability in mixes containing about 6 percent asphalt, regardless of method of test.

7. The void content of a mix is affected to a higher degree by the asphalt content than by the type or quantity of filler contained in a mix.

8. Pulverized loess as a filler yields higher void contents than equal quantities of lime dust.

9. The cohesion of a mix generally increases with increase of filler above 5 percent.

10. The trafficability of a mix generally improves with increase of filler up to an optimum around 7 percent, provided the amount of filler is consistent with asphalt content.

11. Mixes using softer aggregates tend to be displaced under traffic less than mixes with harder aggregates.

12. Pulverized loess can be used effectively as a filler in high-type asphaltic mixes when mixes are properly designed.

13. The addition of asbestos to a mix has varied effects on the stability and voids of mixes containing various aggregates, and therefore, each mix must be evaluated independently.

14. The addition of asbestos to a mix has a decided beneficial effect on cohesion and trafficability.

15. In the design of mixes for heavily traveled roads some consideration must be given to the trafficability of the mix, because there seems to be no correlation between stability and trafficability.

ACKNOWLEDGMENTS

This investigation was authorized and sponsored by the Iowa Highway Research Board of the Iowa Highway Commission under Project No. H.R. 79. The work was done by the staff of the Bituminous Research Laboratory, Engineering Experiment Station, Iowa State University in cooperation with engineers of the Iowa Highway Commission who selected the location of the paving project sites, arranged for procurement of the necessary materials from the sites, and provided the necessary field information from time to time.

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Cold-Mix Construction Considerations With Cutback Asphalt

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•THE TERM "cold mix" is normally used in Kansas to describe the mixing process of a soil and/or an aggregate with a cutback asphalt without heating the aggregate except by the Sun. In general this process involves some or all of the following seven steps:

1. Initially placing a windrow of aggregate on the roadway to be surfaced;
2. Reducing the moisture content to a suitable value by blade mixing;
3. Equalizing it to a uniform cross section;
4. Adding a cutback asphalt by means of suitable mixing and/or distributing equipment; (The previous steps can be eliminated if nornal plant-mix methods are employed utilizing a dryer.)
5. Manipulating the material with blades and tillers to aerate the volatiles, further reduce the moisture content, and thoroughly mix the combination;
6. Spreading it over the roadway in thin lifts; and
7. Compacting it by suitable rolling equipment.

There are, however, other problems that result from the aggregate selected, the types of equipment used, and the prevailing weather during the course of construction. These are the problems with which this paper is concerned.

AGGREGATE SELECTION

Of primary importance is the availability of suitable material in the immediate vicinity of the proposed project. A thorough knowledge of prospective locations and related characteristics enables economic aggregate selection, reduction of unexpected aggregate problems, and accurate engineering estimates.

Most Economic Aggregate Selection

The physical characteristics of the material, such as gradation, plasticity, wear, soundness. absorption, and specific gravity, dictate the type of surface design, probable thickness based on given conditions, and the economics of its use. The criteria for the determination of the relative worth of a particular material with regard to these properties for use in bituminous stabilized mixes have been reasonably well established in Kansas as it has elsewhere. Existing specifications (3) differentiate between the best cold-mix material and the poorest by varying properties to fit local aggregate conditions. In general, revisions to the specifications vary only the gradation, screen spread, and plasticity index (PI) for all classes of cold-mix material while maintaining other quality requirements at a constant value. The decision to be made with regard to use of a local deposit must be based on its economy in relation to the thickness needed for a given traffic volume and the amount of asphalt required. This must be compared to the use of less thickness of a higher grade material with a resultant saving in asphalt but increased cost in material and, possibly, transportation if the material is not local in origin.

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In Kansas the determination of surface thickness is predicated on the triaxial method of design (1), which has been used continuously since 1945 and includes the testing of each component of the roadbed structure. The subgrade and the proposed surfacing materials are each tested separately to determine the thickness of the selected surface course through the use of the following modification of a formula presented by Palmer and Barber (2):

$$T = \sqrt{\left(\frac{3Pmn}{2CS}\right)^2 - a^2}\ \sqrt[3]{\frac{C}{C_p}} \tag{1}$$

in which

T = thickness required,
C_p = modulus of deformation of surface course,
C = modulus of deformation of subgrade or subbase,
P = basic wheel load (single wheels),
m = traffic coefficient based on volume,
n = saturation coefficient based on rainfall,
a = radius of area of tire contact corresponding to P and m, and
S = permitted deflection of surface.

The stress-strain curve is used for determining the modulus of deformation of the material being tested and is the secant modulus between the two points on the stress-strain curve limiting the range of stress determined or the stress difference divided by the strain. Typical triaxial modulus of deformation values for the surfacing materials used in Kansas range from 5,000 psi for the fine-graded, cutback-asphalt stabilized-base courses to 25,000 psi for the most densely graded surface courses mixed with an asphalt cement.

The traffic coefficient, m, is assigned a value in accordance with anticipated traffic volume as forecast by traffic studies for the number of years the pavement is designed to serve without heavy maintenance. It varies in increments of $\frac{1}{8}$ from a minimum value of $\frac{1}{2}$ corresponding to a traffic volume of 50 to 400 veh/day to a maximum value of $\frac{12}{8}$ representing 13,501 to 20,000 veh/day.

The saturation coefficient, n, is based on the average annual precipitation in the area of the proposed project. Inasmuch as all designs are based on test results from saturated specimens, the design formula must be modified by the appropriate coefficient. This value varies in increments of 0.1 from a minimum of 0.5 corresponding to a precipitation range of 15.0 to 19.5 in. in the western part of the State to a maximum of 1.0 representing 40.0 to 50.0 in. in the southeastern counties.

With this system, the structural thickness of any type of flexible surface course, base course, or subbase may be computed, once the basic data have been obtained from tests, by means of the following formula when the term, T, from the basic formula has been computed:

$$t_t = (T - t_p)\sqrt[3]{\frac{C_p}{C_t}} \tag{2}$$

in which

t_t = thickness of base course or subbase,
T = thickness of flexible pavement required directly on the subgrade,
t_p = thickness of flexible pavement desired in the combination,
C_p = modulus of deformation of flexible pavement, and
C_t = modulus of deformation of base course or subbase.

TABLE 1

EQUIVALENT THICKNESSES AND TRIAXIAL MODULUS VALUES OF VARIOUS BITUMINOUS STABILIZED MIXES

Surface Course	C_p (psi)	T (in.)
Hot mix	25,000	6
Cold mix:		
Dense-graded	15,000	7
Fine-graded	5,000	10

This formula establishes the relationships given in Table 1 for various types of presently used bituminous stabilized pavement components.

A number of specifications has been adopted in Kansas (3) to fit the wide range of materials utilized in both cold-mix and hot-mix asphalt stabilization projects. The specifications considered here are those presently used for the construction of new surfaces or resurfacing projects on both the State and county highway systems, by contract, as well as those dealing with acquisition of bituminous patching aggregate by contract. In addition to specific grading bands or limits, provisions have also been included to allow the use of special aggregates such as dune sand or other economical materials occurring on or adjacent to a project site. In many instances such an aggregate exists at a particular location and is well suited to a road mix but is not universal enough to be included as a standard specification. Efforts have also been made to determine how and where some of the marginal materials, such as those which have a high PI or a high absorption value, may be used.

Reduction of Unexpected Aggregate Problems

The physical properties and the extent of the deposit can be determined by obtaining samples from pits or probe holes and submitting them to the laboratory for analysis. However, to date, only by experience can the engineer or the contractor know the problems which a given material may present as a result of characteristics not always measurable by the laboratory or controlled by specifications. One of the most important of these concerns the degree of pulverization of the mix that is obtainable with ordinary mixing and pulverizing equipment. The result of pulverization is the production of "effective fines," generally considered to be that portion passing the No. 200 sieve. Material of this size is necessary to obtain stability, and fines introduced as dry, hard clods or tightly cemented to the coarse aggregate will not produce effective fines by normal mixing methods. The present Kansas Highway specifications require that a percentage of the total minus No. 200 mesh material as determined by washing and dry screening pass the No. 200 mesh sieve by dry screening before washing. Because these characteristics are not always apparent without moisture, the material may appear highly satisfactory if the deposit is dry and may meet the gradation and PI requirements, but if subjected to a series of rains before addition of asphalt, the problem of pulverization may be of utmost concern. This is usually brought about by an aggregate which has a thin, highly plastic clay coating on the coarse aggregate to which the granular fines adhere. In wash tests, both the clay coating and the granular fines appear as effective fines, whereas the PI of the clay coating may considerably exceed the specification value. Because the PI is determined by percentage of material which passes the No. 40 mesh sieve, that of the clay may be reduced to an acceptable figure if sufficient fine sand is present in the material. This possibility is not indicated by the PI nor can these conditions be readily duplicated in the laboratory. Some of the detrimental aspects of this condition are as follows:

1. The necessity for and the cost of adding additional mineral filler because the minus No. 200 mesh material in the mix cannot be made effective by normal means of pulverization;
2. The cost of repairing failures due to the stripping of the asphalt from the coarse aggregate, because it is coated with clay and fines which prevents the asphalt from directly contacting its surface; and
3. The "raveling" and degradation of the riding surface of the roadway resulting from the loss of aggregate by the poor bond between the coarse and fine particles.

Accurate Engineering Estimates

A thorough knowledge of local aggregate deposits and their characteristics provides the only means of making an accurate engineering estimate of the cost of a proposed project for budgeting or bidding purposes.

The attempt has been made to emphasize aggregate selection as a major consideration in the construction of any asphalt-stabilized surface.

MIX DESIGN

The first cold-mix asphalt surface in Kansas was constructed on US 50 in Stafford County near Zenith in 1930. It was constructed 3 in. in depth and 22 ft wide and performed very satisfactorily with normal maintenance for 28 yr. It was constructed in part from gravel existing on the roadway plus a graded washed gravel, a pit-run sand, and volcanic ash as a mineral filler. In general, the mix was considerably coarser than any cold-mix mat or base built under present Kansas specifications. Both laboratory and field results indicated an average of 35 percent of the material retained on the No. 8 sieve with 5.5 percent passing the No. 200 sieve. Current specifications for cold-mix surface course material in Kansas allow a minimum of 25 percent retained on the No. 8 sieve, whereas some of the bituminous-stabilized bases require only 15 percent. These reductions in over-all coarseness of present-day specifications were made to adjust them to more nearly fit existing aggregates for economy as well as to conserve deposits of material.

The information used in the construction of the first cold-mix surface was obtained from New Mexico, and present-day methods of mix design and determination of the proper asphalt content have changed very little. From time to time changes have been made in grading requirements, but this was done, primarily, to make the specifications meet the gradation of an existing aggregate deposit rather than to improve the stability or life of the mix. Recently, however, increased emphasis has been placed on the effective fines aspect of each specification with a marked improvement in the immediate stability of the compacted surface. This does not imply that no attempt has been made to improve the design method but rather that very little has been learned which would make any radical or obvious change.

Kansas was one of the first States to adopt the triaxial compression test for the determination of rigid and flexible pavement thicknesses and of stability of soils. It was on the basis of results obtained from triaxial tests that the PI was limited to a maximum value of 6 for the individual aggregates as well as the combined material for bituminous mats. This PI value was determined from a comparison of the moduli of deformation of specimens molded from bituminous-stabilized cold-mix materials submitted to the laboratory from projects throughout the State. In each case, a minimum of two specimens were molded, and one was saturated with water before testing. In any mix with a PI value greater than 6, the modulus of deformation of the saturated sample was greatly reduced as compared to the sample which was not saturated.

Efforts have been made for years to use the modulus of deformation as an indication of optimum asphalt content in bituminous-stabilized mixes, but, in general, no peak occurs. Instead, a very flat curve results which cannot be utilized to indicate this point, but if extended by additional samples at higher asphalt percentages, a sharp drop in the curve takes place indicating the maximum asphalt content which can be tolerated in a particular mix. A comparison of the moduli of deformation values (Table 2) provides a means for increasing or decreasing the proposed surface thickness based on results obtained from cold-mix specimens tested over a period of years.

Attempts have also been made to adapt Marshall equipment to cold-mix design problems without much success even though the specimens were tested at room temperature. It was necessary to modify the practice of placing Marshall specimens in a 140 F waterbath for 20 min before testing because specimens molded with cutback asphalts will either slump or show no stability at this temperature.

Uniform aeration of the volatile portion of a bituminous material stabilized with cutback asphalt is of major importance in all mix stability determinations in the laboratory. Because the volatiles act as lubricants, the aeration time must be increased with

TABLE 2

TRIAXIAL MODULUS VALUES FOR TYPICAL GRADINGS[a]

Type	Percent Retained on Sieve: Inch			Number									Asphalt Content[b] (%)	Density[c] (pcf)[3]	Triaxial Modulus 100 F (psi)
	1	3/4	3/8	4	8	16	30	40	50	80	100	200			
BC-1	0	2	8	15	27	41	54	63	73	83	85	89	4.2	131.2	6847
BC-1	0	5	17	21	31	47	66	75	82	87	88	89	3.6	133.0	6973
BC-4	0	5	9	14	22	36	54	65	77	88	89	92	3.3	129.7	5343
BC-4	0	0	3	8	21	38	57	—	76	85	87	90	3.8	130.8	5483
BC-5	0	0	1	6	18	36	54	65	77	86	87	88	3.9	131.6	4800
BC-5	0	0	2	7	21	42	59	69	79	85	86	88	3.3	132.5	4927

[a]Samples were removed from windrows at completion of aeration and mixing. All were mixed with MC-4 grade cutback asphalt.
[b]Based on dry weight of aggregate and determined by a reflux-type extractor.
[c]Value is 95 percent of Marshall density obtained on same material (50 blows to each side of specimen at 140 F).

the asphalt percentage to maintain a constant relationship of the percent volatiles retained between all specimens. This is difficult to accomplish with small samples where an oven is used as a heat source and where the volatiles retained must be determined by distillation. It is probable that this is a part of the answer for the variations which result in using either the triaxial or Marshall equipment to determine the stability of a mix.

For the bituminous-stabilized surface and base courses utilizing cutback asphalt which are constructed under present Kansas specifications, the use of either or a combination of the results of the Nebraska or the New Mexico formulas (Appendix) generally provides a reasonable starting point for field determination of the proper asphalt content. These formulas are based on the surface area of the material. For extremely fine mixes, the only source of information as to asphalt content is previous experience with these materials because they are too fine for either formula to give a satisfactory answer. Either of these formulas could possibly be corrected to make suitable allowance for the increased surface area.

The preceding discussion indicates that very little guidance or positive answers to cold-mix design problems can be given by the laboratory with regard to the optimum asphalt content.

CONTROL OF MATERIALS

The success of any well-designed surface or base course depends on the uniformity of construction, particularly with regard to the materials involved. The two components, aggregate and asphalt, present individual problems which are considered separately in the following discussion:

Aggregate Distribution

The fundamental concept of most multiple-aggregate mix designs is based on the various materials being combined on a weight basis. The actual methods employed for distributing aggregates in their proper proportions are well known by anyone who has ever been concerned with this type of construction; however, they are rather crude as are many other aspects of cold-mix construction.

In asphalt-stabilized mixes gradation is usually considered to contribute a major part to the stability. Stability is generally conceded also to be some measure of the load-carrying capacity of a surface. Each type of asphalt-stabilized mat or base has a particular load-carrying ability which is used in determining the thickness requirements for a particular proposed surface. As a result, it becomes imperative that if uniform stability is to prevail, every means within reason must be employed by both engineer and contractor to produce the material uniformly from the deposit and combine it on the road as accurately as possible. This can only be done by cooperation on the part of both parties involved, and by an understanding of each others problems.

The engineer must accurately sample the materials which the contractor has selected for use and determine their physical properties before a combination can be calculated which will meet the specifications.

The engineer must keep in mind at all times that the contractor had obviously made some preliminary calculations using these same aggregates in order to establish a bid price and that his success or failure, economically, depends to a certain extent on using these same percentages. The greatest percentage of the cheaper material should always be used in mix calculations unless it will impair the quality of the mix or a mutual agreement concerning the addition of a more expensive material has been reached with the contractor.

The lower cost material is generally a pit-run aggregate, and although cheap to produce, it could be more expensive in the end for the contractor unless the deposit is very uniform in grading and is worked properly. Once the various percentages of the individual aggregates have been determined, it becomes the contractor's responsibility to continue to produce material with uniformity and of the same quality and grading.

Some of the combined windrow samples may fail to meet the specification requirements unless the material is thoroughly mixed at the pit, a fairly elaborate screening plant is used to process the aggregate, and/or exceptional care is taken in placing the individual aggregates on the roadway. An additional problem results from the tendency to produce individual materials with only the minimum amount of coarse aggregate required to meet the lower specification limits because of the cost involved.

The cost of additional material to correct any discrepancy, determined by windrow sampling, must be borne by the contractor if the surface being constructed is one in which the total plan thickness is being laid in one lift, or the tonnage already on the road equals plan quantities. A representative windrow sample must be obtained to prevent a serious loss to the contractor of both time and money. If the engineer is satisfied in all respects that the material does not meet specifications, it should be determined whether the error is of sufficient magnitude to require correction. At this point it is well to know that specification limits were established: (a) to provide a means of control, (b) to maintain a gradation range which will produce a stability satisfactory for design thickness, (c) to fit local aggregates, and (d) to provide a common basis for competitive bidding in the case of projects constructed by contract. If the deficiency can be justified on the basis of these reasons, the contractor should be allowed to proceed. The difficulty of producing and combining the various aggregates into a suitable mixture emphasizes the necessity for the cooperation of all personnel engaged in this type of construction.

Asphalt Distribution

Whether the asphalt percentage calculated for a particular mix has been determined by the laboratory, by formula in the field, or from experience alone, control of asphalt content must be as strict as possible. The argument of whether the asphalt content is too high or too low for a particular mix will probably continue until an absolutely accurate means for its determination under all conditions has been found. In any case, close supervision of a number of operations is a necessity to produce a finished surface uniform in asphalt content. Areas or sections with an excess of asphalt must be removed because a surface constructed of this material is unstable and will eventually corrugate or rut.

The windrow of material must first be absolutely uniform in cross-sectional area, as well as gradation. This condition is a necessity for proper operation of most equipment used to add asphalt to the aggregate. The types of machines used may be the ordinary asphalt distributor, a traveling plant which picks up the entire windrow and runs it through a closed pugmill where the asphalt is introduced, an open-bottomed traveling pugmill which straddles the windrow and adds the asphalt during the mixing process, or a rotary tiller-type mixer with an attached spray bar. In all cases, constant speed on the part of the machine is absolutely necessary to obtain a consistent asphalt percentage. If constant speed is not maintained, sections with an excess of asphalt will be produced. Speed is most often interrupted by an extremely large windrow which causes the machine to slow down momentarily due to overload.

These machines are, in general, equipped with positive displacement asphalt pumps which deliver constant volume at any constant pump speed; however, asphalt content is calculated on the basis of a percent of the dry weight of the aggregate. This makes it necessary to have an accurate calibration of the pump output through its speed range in order to make corrections for the variations in temperature of the asphalt used during the course of construction.

MIXING

Aggregate and asphalt mixtures are generally either road mixes, plant mixes, or a combination of both procedures. Because each method has its own unique problems, they must be considered separately.

Road Mix

Nearly all cold-mix material in Kansas is mixed on the roadway by motor graders and rotary tillers. The initial mixing is usually accomplished by the particular machine used to add the asphalt to the windrowed material, but it may be added to a uniform lift (spread over the roadway) by a conventional asphalt distributor in not less than three applications per inch of base thickness. Initial mixing, if properly performed, can be the most effective period of all. However, it is often felt that as long as the proper amount of asphalt is distributed per unit length of windrow, the blades and rotary tillers will break up the oil balls and complete the mixing. When the weather is extremely hot and dry, this procedure generally works satisfactorily, but the question arises as to whether a black windrow is evidence of satisfactory mixing. By past and present standards satisfactory mixing is gaged only by visual inspection based on uniform color; however, some of the coarse aggregate which is always difficult to coat may only be discolored and not have a sufficient asphalt film thickness to provide a satisfactory bond. The asphalt is at its highest temperature during this particular part of the mixing phase, and as a result, its viscosity is lowest. Lowered viscosity results in a reduced film thickness, which allows a greater number of aggregate particles to be coated. A windrow of a size which allows all material to be agitated by the pugmill blades will further the success of this operation.

In addition to coating all aggregate particles throughout the entire windrow, the mixing phase releases the volatile portion of the cutback asphalt and the moisture. The cutbacks generally used in Kansas for road-mix construction are medium curing type. The diluent added to the base stock serves to lower its viscosity, allowing it to be fluid at normal summer temperatures. A portion of this diluent or volatiles must be removed by aeration before the asphalt will become tacky enough to develop a bond between aggregate particles sufficient for mix stability. For practically all road-mix construction in Kansas, an MC-4 grade of cutback asphalt is specified. The present specifications limit the diluent to a maximum of 15 percent by vol and require that at least 30 percent by wt of the volatiles present in the cutback asphalt at the time of mixing with the aggregate be removed before the actual laying and compacting of the mix can begin.

Because the percentage of volatiles in cutback asphalts can vary widely, it may be more desirable to limit the amount of volatiles which may remain in a mix rather than to specify the percent which must be removed. The percentage selected must be less than any volatile content supplied by the cutback producers to be any more significant than that established by the present specifications inasmuch as all grades of cutback asphalt will require some aeration.

Over a period of years, the specifications for cutback asphalts have become more stringent. This has been accomplished by more rigid controls of the distillation requirements affecting the diluents that may be used and by limiting the variation of penetration of the base stock from any one producer to 50 points during a construction season. These changes have produced more consistent results from project to project where the cutback is supplied by different producers.

Road-mixed projects, quite frequently, are also plagued by the weather. The specifications allow asphalt to be added to the windrowed material when the moisture content is 4 percent or less of the dry weight of the aggregate for predominantly sand mix or 3 percent plus one half the moisture absorption of the aggregate for other aggregates. After the asphalt has been added, the mix cannot be laid until the moisture content is less than 1 percent of the total weight of the mix. When projects are subjected to a series of rains, the cost of drying out the windrow each time quickly reduces the profit in the job for the contractor. In many instances additional material must be hauled to supplement that which was washed away by a torrential rain.

Each time the windrow is aerated to reduce the moisture content, volatiles are released. This condition is desirable initially but may be detrimental to the success of the job if enough volatiles are lost to make the mix brittle and difficult to lay. Rain and cold weather make road mixing hazardous in cost to the contractor and in the quality of the finished product.

Plant Mix

Although plant-mixed cold-laid materials are rapidly becoming common place throughout the highway industry, Kansas has had relatively little long-term experience in this field.

Two projects have been built utilizing the continuous-mix type of hot-mix plant; one approximately 15 yr ago on the county secondary system and the other nearly 10 yr ago on the State system. With the advent of the Interstate Highway System, this type of construction has been utilized for bituminous-stabilized base course material on approximately 20 mi of Interstate 70 in Trego and Gove Counties during the past 3 yr. These were not cold mixed, however, because the aggregates were heated, but a cutback asphalt was used as the bitumen. This type of mix must be further subdivided by the method employed in lay-down operations.

Plant Mixed (Finisher Laid). —The first plant-mixed mat was constructed in May 1948 on Halstead Street at the east edge of Hutchinson, Kansas, as a county secondary project. It utilized a BMA-1 type of aggregate (similar in grading to the BC-1 specification) and MC-5 cutback asphalt and was laid 3 in. thick. The asphalt and the aggregate were both heated to 250 F. The New Mexico formula was used to determine the asphalt content, calculated to be 5 percent of dry weight of aggregate. The mixed material was placed on the roadway with a conventional laying machine, but the contractor was unable to compact the mix immediately because of its fluffy condition caused by the volatiles remaining in the mix. Various types of equipment were used to manipulate and aerate the material. After 2 days the material became tacky and was compacted by pneumatic-tired and flat-face rollers. Aeration checks during construction indicated a loss of only 14 percent of the volatiles. Repeated aeration checks made during June, July and August indicated a maximum loss of 23 percent of the volatiles contained when shipped from the refinery. This project has been highly successful and has required very little maintenance although most traffic is of heavy industrial type.

The second project was constructed in Rush County on US 183 within LaCrosse, Kansas, in 1953. An aggregate meeting the AA (Special) requirements (similar in grading to the BC-4 specification) and an MC-5 cutback asphalt were mixed and laid 6 in. thick by a conventional laying machine. This mix acted much the same as the previous project and required approximately 2 days of manipulation by a farm cultivator before it became tacky enough to be compacted. It has required relatively little maintenance and has satisfactorily carried both local and through traffic of approximately 1,700 veh/day.

It is evident from these two projects that the heating of both aggregate and cutback asphalt does not eliminate the need for additional aeration of the volatile fraction before compaction can begin.

Plant Mixed (Blade Laid). —The plant-mixed cold-laid material used on a portion of Interstate 70 differed primarily from the previously discussed project in that the plant-mixed material was hauled to the job site, dumped in a windrow, and blade mixed and laid by means of motor graders rather than the conventional bituminous pavement finisher. However, an automatic batch-type hot-mix plant and/or a stabilization mixer with conveyor-connected dryer were used for blending and mixing the aggregate and asphalt. This type of operation lends itself to high-volume production in keeping with the quantities involved in the usual interstate project. This system eliminated the problem of aeration encountered with the previously described projects and alleviated the drying hazard normally associated with road mixing.

LAYING AND COMPACTING

Though all other problems have been dealt with successfully, the final test of any type of roadway surface is how it rides. Most cold-mix asphalt in Kansas in laid by a motor grader. The subgrade, base or existing surface on which the mix is being laid dictates how smooth the finished surface is unless crown and longitudinal irregularities are corrected. Localized or spot leveling plus a leveling course to correct the crown is generally standard practice before the final lift is applied.

Until recently, the success of any motor grader-laid surface depended entirely on the skill of the operator. This situation was improved by the adaption of an electronically operated, constant slope control device to the blade of a motor grader. By means of this control the surface may be trimmed quite accurately to conform to any desired crown.

It has been suggested that a compaction requirement should be established for cold-mix laid base and mats to assure a uniform density in the finished surface as required in hot-mix specifications. Information from laboratory tests conducted at Oklahoma State University (4) indicates that it may not be desirable to compact materials stabilized with cutback asphalts to a particular density requirement unless the amount of moisture and volatiles can be accurately controlled. Tests indicate that maximum stability and maximum density do not occur at the same moisture-volatile point. Any control of the compaction of a cutback asphalt-aggregate mixture by a density requirement must be accompanied by a limitation in the amount of water and volatiles in the mixture. These results may not be applicable in the field and to Kansas mix designs, but they indicate that other agencies are interested in the same problems.

APPLICATION

The simplicity of production and economy of construction makes cold-mix material adaptable to a wide range of applications and appealing to many engineers and contractors. This type of asphalt stabilization is commonly used for bituminous surfacing, resurfacing, base, and stabilized shoulders, and for maintenance patching.

The reasons for the selection or consideration of cold mix vs hot mix may be any of the following:

1. Construction is more economical;
2. Small volume needed makes plant-mix prohibitive in cost;
3. Equipment normally used for routine highway maintenance is all that is normally required to produce cold mix;
4. New construction involving realignment may permit road mix without traffic interference;
5. A relatively wide range of temperatures can be tolerated during construction without detrimental effects;
6. Many marginal aggregates may be acceptable for a road mix but unsuitable for plant production; or
7. Maintenance patching material may be prepared in advance and stockpiled until needed with little danger of deterioration or loss.

USE AND COSTS

In Kansas, the use of cold-mixed materials for base and surface courses prepared by road mixing is on a steady decline in favor of plant-mixed materials whenever possible. This applies not only to surfacing projects being constructed by contract but also to maintenance work performed by State Highway personnel. Such a change has been the result of an effort to reduce the weather hazard always present during road-mixing operations. A comparison of the road-mixed and plant-mixed material costs

TABLE 3

ANNUAL TOTAL OF COLD-MIXED AND HOT-MIXED ASPHALT STABILIZED MATERIALS USED IN KANSAS

Year	Cold Mix				Hot Mix			
	MC-4 Asphalt BC-Type Agg.[a]		MC-3 Asphalt BPA-Type Agg.[b]		AC-7 Asphalt BC-Type Agg.[a]		AC-5 Asphalt HM-Type Agg.[a]	
	Tons	Cost[c] ($/ton)	Tons	Cost ($/ton)	Tons	Cost ($/ton)	Tons	Cost ($/ton)
1960	1,146,072	2.45	—	—	597,112	3.47	536,456	5.82
1961	862,893	2.47	—	—	756,597	3.60	416,393	5.63
1962	504,570	2.40	1,000,000	4.06	1,358,257	3.65	419,831	4.98

[a]Used for surfacing and resurfacing projects by contract.
[b]Used by Maintenance Department.
[c]Includes manipulation and lay-down costs.

and quantities utilized in the construction and maintenance of the Kansas Highway System by contract indicates this change and provides a means of estimating the expense involved. These values are given in Table 3. The data, which have been correlated from laboratory tests, do not indicate any gain in stability to justify the increased cost of using plant-mixed materials with an asphalt cement as compared to the same aggregate mixed on the road with a cutback asphalt. However, in many cases, this additional cost can be justified where a roadway must be resurfaced under traffic, thereby reducing the traffic hazard for the workmen and relieving the congestion by needing less equipment as well as eliminating the windrow of material.

Although the use of cold mix is decreasing each year, Table 3 indicates that it is still a major construction item in the Kansas Highway Program.

REFERENCES

1. State Highway Commission of Kansas, "Design of Flexible Pavements Using the Triaxial Compression Test." HRB Bull. 8 (1947).
2. Palmer, L, A., and Barber, E. S., "Soil Displacement Under a Circular Loaded Area." Proc. HRB, 20: 279-286 (1940).
3. State Highway Commission of Kansas, "Standard Specifications for State Road and Bridge Construction." (1960).
4. Herrin, M., "Drying Phase of Soil-Asphalt Construction." HRB Bull. 204, pp. 9-10 (1958).

Appendix

ASPHALT PROPORTIONING FORMULAS

Formula A (New Mexico)

$$P = 0.02a + 0.07b + 0.15c + 0.20d$$

in which

P = percent of bituminous material by weight of dry aggregate,
a = material retained on No. 50 sieve in percent,
b = material between No. 50 and No. 100 sieve in percent,
c = material between No. 100 and No. 200 sieve in percent, and
d = material passing No. 200 sieve in percent.

Formula B (Nebraska)

$$P = AG\,(0.02a) + 0.06b + 0.10c + Sd$$

in which

P = percent of bituminous material by weight of dry aggregate;
A = absorption factor, equal to 1 for sand-gravel and to $1 + \frac{2}{3}$ the approximate net water absorption of other aggregates;
G = gravity correction factor, equal to 1 for sand-gravel and inversely proportional to other average specific gravities on the base of 2.61 as the gravity of river sand-gravel (retained on No. 50 sieve);
a = material retained on No. 50 sieve in percent;
b = material retained between No. 50 and No. 100 sieve in percent;
c = material retained between No. 100 and No. 200 sieve in percent, plus c_1;
d = percent of fines determined by average of percent passing the No. 200 sieve dry screened and that passing the No. 200 on wash test in separate determinations;
c_1 = percent of material equal to difference between percent of material passing No. 200 sieve on wash test and d; and
S = 0.2 except when volcanic ash is used as a mineral filler, in which case S shall be determined by laboratory tests.

The quantity, d, in the Nebraska formula shall be found by separate determination on two similar samples of combined material; one sample shall be dry screened only, and the other sample shall be tested by the wash test plus dry screening after washing.

In both formulas, cutback oils of MC and RC types should be increased in quantity by the amount of diluent which is expected to be lost during aeration and curing.

Effect of Moisture on Bituminous Pavement In Rocky Mountain Areas

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The paper deals with problems in Colorado, New Mexico, Utah and Wyoming resulting from the preferential affinity of many locally available aggregates for water rather than for asphalt. Surface raveling and, in extreme cases, softening and complete disintegration of the bituminous pavement result. The characteristic is commonly referred to as "stripping," meaning that the asphalt coating comes off the aggregate in the presence of water and is replaced by the water. It is more pronounced with the lighter types of asphaltic materials, such as cutbacks, used in road mixes and has been one of the principal reasons for the general replacement of road mixes by hot plant mixes and asphaltic concrete using asphaltic cements. Even with higher types of pavements the problem is common enough and serious enough to require the general use of seal coats, either immediately or after a period of several years. Types of seal coats used are described. Another common approach to the problem has been the use of chemical additives and, more recently, hydrated lime. Their effect and methods of specifying and use are described.

Early realization of the stripping problem led to the development of the immersion-compression test (AASHO Designation T-165) for measuring the effect of water on compacted bituminous mixtures and of the static-immersion test (AASHO Designation T-182) for determining the effect of water on coated coarse aggregate particles (used in surface treatment and seal coats).

The paper is not intended to be a technical analysis of the chemistry or mechanics of the stripping action, nor does it present any guaranteed solutions. It does, however, point up the extent and seriousness of the problem with the hope that it will stimulate further research leading to more consistently water-resistant pavements.

•THE PROBLEM of the effect of water on bituminous paving mixtures is certainly not limited to the Rocky Mountain area (1, 2). Ideally, water should have no effect on pavement, but all too often adhesion has been reduced between the asphalt and aggregate to the point that serious "stripping" has occurred with resultant loss of mat stability and often severe "raveling" of aggregate from the surface. Stripping is defined as the loss of asphalt films from aggregate surfaces in the presence of moisture (9), and raveling as the loss of aggregate particles in the surface of the pavement—usually caused by loss of adhesion between aggregate and asphalt. This action is more pronounced in the road-mix types but has even occurred to a serious extent in the hot plant-mix and asphaltic concrete types. The Rocky Mountain area is generally dryer than other parts of the country so it might be expected the problem would be less severe than elsewhere. However, this lack of moisture may well make the effect of water more pronounced when it does become available. Water has not been available to leach out deleterious

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fines or disintegrate soft particles in natural deposits of aggregates, as is the case in areas of greater rainfall. Possibly because of this, degradation is common with many of the aggregates available in the area. Although this degradation seldom increases plasticity significantly, it undoubtedly helps to reduce resistance of the pavement to the effects of water.

This problem may be common because of the widespread use of local, on-the-job aggregate deposits rather than those commercially produced. Specifications have commonly been written to fit these local aggregate sources rather than to require a high-quality standard aggregate.

In recent years, especially since the start of the Interstate program, there has been a general upgrading of specification requirements with the result that some aggregates formerly permitted are now excluded or, if permitted, are upgraded by improved processing (as pre-wasting of fines and use of screening equipment which removes and wastes coatings on the aggregate and breaks down and wastes softer aggregate particles) or by the addition of mineral filler, asphalt, cement, or hydrated lime. Washing of aggregates for bituminous construction has not yet come into general use, but as specification standards become higher and the availability of naturally high-grade aggregate becomes less, this will be a logical development.

EARLY EXPERIENCE

The early bituminous pavements in rural areas of the Rocky Mountains were nearly all of the road-mix type using slow-curing road oils and then cutbacks of the MC-types. It was commonly recognized that these surfaces lacked resistance to the effects of moisture, and seal coats were used as deterrents. In many cases, however, water still penetrated the pavement—probably by capillary action or as water vapor. Excess water in the mat was a common cause of early deterioration. Observations of certain pavements in Utah and Colorado in 1949 (3) indicated that all bituminous pavements containing more than 2 percent moisture failed and that the moisture content increased as the percentage of minus No. 200 aggregate increased. The maximum allowable minus No. 200 size appeared to be 12 percent. The tests did not indicate the Plasticity Index (PI) of the mat aggregate, higher PI values were probably associated with higher moisture contents with pavement failures.

Although seal coats prevent or retard surface raveling, they have not been effective, and occasionally have been detrimental, in preventing stripping within the mat. The general use of seal coats on road-mix mats seems to have carried over into use on hot plant-mix and bituminous concrete surfaces. Only in recent years has there been any tendency to leave seal coats off these higher type pavements and often then with reluctance and a common feeling that they will need seal coats within a short time. Too often this feeling has been borne out.

The seriousness of the problem was first brought forcibly to many people's attention by an experimental project in Colorado in 1941. Sections of road-mix bituminous surfacing using different sources and types of asphaltic materials were constructed with a local aggregate of quality adequate by the normally accepted standards at that time. However, all sections quickly showed serious distress with raveling taking place with the first rain. This necessitated prompt seal coating of all sections, thus obscuring any differences in the sections. A subsequent laboratory study of the aggregate by the Bureau of Public Roads Materials Research Division (4) led to the development of the immersion-compression test (AASHO Designation T-165 and ASTM Designation D-1075).

Even before the Colorado experiment there were some very unhappy experiences with asphalt stripping. At one time in the mid 30's, open-graded bituminous mat was tried on direct Federal construction projects. Because most aggregate sources were gravel, to obtain the 100 percent crushed aggregate required, the natural pit fines were wasted and the required aggregate was produced by crushing the oversized gravel. This gave a mat of high mechanical stability, but with high void space. To keep surface water out, the surface was choked with fines and seal coated. The aggregates were mainly granitic and although surface water may have been kept out and the surface looked good, the asphalt soon stripped badly from the coarse crushed aggregate in the lower parts of the mat, and these aggregate particles became coated with water instead of asphalt. The general opinion at the time was that the seal was ineffective and allowed surface water to enter and soften the mat. This type of construction was discontinued in favor of a return to the dense-graded type. However, there is no assurance that free water actually entered the mat from the surface. The open grading and the much greater affinity of the aggregate for water than for asphalt created conditions favorable to the development of stripping. The extreme temperature differentials in the high mountains could well have led to condensation of moisture in the large void space of the open-graded mat.

On one of the projects referred to previously, the State used the wasted pit fines to build a dense-graded road-mix mat on an adjacent project with excellent results. This does not prove that open-graded crushed aggregate mats are necessarily all bad and dense-graded ones all good, but rather that with hydrophilic aggregates the chances for moisture to penetrate and cause stripping and softening of the mat are much less with the denser mixture.

These projects were all built before the days of the immersion-compression test (AASHO T-165), the static-immersion test (AASHO T-182), or any other commonly accepted test to measure the effect of water on the aggregate-asphalt combination. In addition, commercial chemical anti-stripping agents, proprietary products that under some conditions greatly increase the affinity between asphalt and aggregate in the presence of water, had not yet come into general use.

IMMERSION-COMPRESSION TEST

For laboratory determination of the effect of moisture on dense-graded hot plant-mixed bituminous mixtures, many States rely heavily on the immersion-compression test and design mixtures having a wet stability of 70 or 75 percent of the dry stability. This ratio generally seems to be a good one for hot-mix or asphaltic concrete, although it is often difficult to achieve without some stabilizing admixture. Swanberg and Hindermann (8) recommended a minimum immersion-compression wet-dry stability of 75 percent. Utah considers the actual wet stability more significant than the wet-dry stability ratio and specifies a minimum wet stability of 150 psi. Wyoming accomplishes this by requiring a minimum dry stability of 250 psi and a wet-dry stability ratio of 70 percent which gives a minimum wet stability of 187 psi. This approach has considerable merit because mixtures of rather clean, coarse-graded aggregates often show a good immersion-compression ratio but may be too low in either dry or wet stability or density to make the best pavement. In addition, chemical additives commonly reduce the actual dry stability value and increase the wet stability; that is, they bring the two values closer together, thereby increasing the wet-dry stability ratio. In this case, the ratio alone does not give a true measure of the nature of the mixture or of the effect of the additives.

In comparing wet-dry stability ratios, the method of compacting and testing the specimens must also be considered. For example, Colorado compacts the specimens with a kneading compactor as used for Hveem stabilometer tests. Specimens so compacted will probably show significantly higher actual stabilities and higher immersion-compression ratios than specimens compacted by the standard double-plunger direct compression method. Some unpublished tests by the Bureau of Public Roads Materials Research Division show that specimens compacted and tested by the standard Marshall method have a wet-dry stability ratio of 112 percent of specimens compacted and tested by the standard T-165 method.

IMPROVED DESIGN AND CONSTRUCTION STANDARDS

In recent years, standards of design and construction of bituminous plant mix and asphaltic cement have been upgraded in this region so that it is believed they are now comparable to standards used elsewhere. Minimum compacted densities are 95 percent of laboratory density (Marshall standard or kneading compactor density) and include air voids of 2 to 6 percent. Sufficient asphalt is used to fill at least 75 percent of the voids in the compacted aggregate. Generally, this requires about 6 percent of 85 to 100

TABLE 1

IMMERSION-COMPRESSION TEST RESULTS[a]

Project or Sample	Additive		Asphalt		Compressive Strength at 77 F		Wet-Dry Stab. Ratio (x 100)
	Type[b]	$\%$[c]	Grade	%	Dry	Wet	
Colo. 2721	None	-	85-100	5.3	456	158	30
Colo. 2721	PC	2	85-100	5.7	444	570	128
Colo. 2721	HL	1	85-100	6.0	378	515	138
Colo. 43076	None	-	85-100	5.8	293	56	16
Colo. 43076	HL	1	85-100	6.3	284	286	101
Utah 169-SA-63	None	-	85-100	5.2	-	195	-
Utah 169-SA-63	HL	1	85-100	5.2	-	283	-
Utah 97-SA-63	None	-	85-100	5.3	-	45	-
Utah 97-SA-63	HL	1	85-100	5.3	-	154	-
Wyo. 2938	None	-	85-100	6.0	302	188	62
Wyo. 2938	None	-	85-100	6.5	309	223	72
Wyo. 2938	HL	2	85-100[d]	6.0	404	404	100
Wyo. 2938	HL	2	85-100[d]	6.5	313	363	116
Wyo. 2938	None	-	85-100[d]	6.0	332	272	82
Wyo. 2938	None	-	85-100[d]	6.5	315	296	94
Wyo. 5120	None	-	85-100[d]	6.0	234	89	38
Wyo. 5120	None	-	85-100[d]	6.5	258	95	37
Wyo. 5120	HL	2	85-100[d]	6.0	285	208	73
Wyo. 5120	HL	2	85-100[d]	6.5	312	204	65
Wyo. 5120	HL	3	85-100[d]	6.0	430	234	54
Wyo. 5120	HL	3	85-100[d]	6.5	425	225	53
Wyo. 5120	HL	4	85-100[d]	6.0	453	275	61
Wyo. 5120	HL	4	85-100[d]	6.5	442	309	70
Wyo. 5120	PC	3	85-100[d]	6.0	280	131	47
Wyo. 5120	PC	3	85-100[d]	6.5	290	143	49
Wyo. 5120	AF	1	85-100[d]	6.5	278	110	39
Wyo. 5120	AF	2	85-100[d]	6.5	283	124	44
Wyo. 5120	FA	2	85-100[d]	6.5	310	80	26
Wyo. 5120	FA	3	85-100[d]	6.5	307	94	31
Wyo. 5120	LWA	2	85-100[d]	6.5	248	67	27
Wyo. 5120	LWA	3	85-100[d]	6.5	243	86	35
Wyo. 62-521	None	-	85-100[d]	6.0	413	102	25
Wyo. 62-521	None	-	85-100[d]	6.5	418	159	38
Wyo. 62-521	HL	2	85-100[d]	6.0	366	255	70
Wyo. 62-521	HL	2	85-100[d]	6.5	344	235	69
Wyo. 62-313-317	None	-	85-100[d]	6.5	296	177	60
Wyo. 62-313-317	None	-	85-100[d]	7.0	305	204	67
Wyo. 62-313-317	HL	2	85-100[d]	6.5	342	310	91
Wyo. 62-313-317	HL	2	85-100[d]	7.0	334	313	94
Wyo. 62-313-317	Cement	2	85-100[d]	6.5	331	283	86
Wyo. 62-313-317	Cement	2	85-100[d]	7.0	305	253	83
Wyo. 62-313-317	None	-	85-100[d]	6.5	299	194	65
Wyo. 62-2778	None	-	85-100[d]	5.25	196	68	35
Wyo. 62-2778	None	-	85-100[d]	6.25	193	105	55
Wyo. 62-2778	None	-	85-100[d]	6.75	190	157	82
Wyo. 62-2778	None	-	85-100	6.25	195	82	42
Wyo. 62-2778	None	-	85-100	6.75	207	115	56
Wyo. 62-2778	HL	2	85-100[d]	6.25	222	179	80
Wyo. 62-2778	HL	2	85-100[d]	6.75	213	231	108
BPR C	None	-	60-70	5.8	183	84	46
BPR C	None	-	85-100	4.9	206	70	34
BPR C	HSA	1	85-100	4.9	143	70	49
BPR C	HL	1	85-100	4.9	277	130	47
BPR C	None	-	85-100	5.6	276	132	47
BPR C	HSA	1	85-100	5.6	250	109	44
BPR C	HSA	1	85-100	6.2	268	120	45
BPR C	HL	1.5	85-100	6.3	296	296	100
BPR D	None	-	85-100	5.6	227	226	100
BPR D	HSA	1	85-100	5.6	212	238	112
BPR D	HL	1	85-100	5.6	225	252	110
BPR E	None	-	MC-3	3.9	70	0	0
BPR E	HSA	1	MC-3	3.9	99	40	40
BPR E	HL	1	MC-3	5.0	95	46	48
BPR E	None	-	120-150 AC	5.8	291	0	0
BPR E	HSA	1	120-150 AC	5.8	290	327	113
BPR E	HL	1	120-150 AC	6.7	315	235	75
BPR F	None	-	MC-3	5.2	62	13	29
BPR F	HSA	1	MC-3	5.2	35	25	45
BPR F	HSA	1	MC-3	5.6	65	27	42
BPR F	None	-	120-150 AC	5.2	211	152	72
BPR F	HSA	1	120-150 AC	5.2	234	194	83
BPR F	HL	1	120-150 AC	5.6	262	219	84
BPR H	None	-	120-150 AC	5.3	374	139	37
BPR H	HSA	1	120-150 AC	5.3	405	307	76
BPR H	HL	1	120-150 AC	6.1	387	367	95
BPR I	None	-	MC-3	4.0	148	33	22
BPR I	HSA	1	MC-3	4.0	111	67	60
BPR I	HL	1.5	MC-3	4.0	184	130	72
BPR I	None	-	120-150 AC	5.2	366	298	81
BPR J	None	-	MC-3	4.7	-	0	0
BPR J	HSA	1	MC-3	4.7	102	55	54
BPR J	HL	1	MC-3	4.7	118	64	54
BPR J	HL	1.5	MC-3	4.7	147	94	64
BPR K	None	-	MC-3	3.7	126	33	26
BPR K	HL	1	MC-3	3.7	187	130	70
BPR K	None	-	120-150 AC	5.7	259	131	51
BPR K	HL	1	120-150 AC	5.7	286	225	79
BPR L	None	-	MC-3	4.4	175	15	9
BPR L	HSA	1	MC-3	4.4	192	35	18
BPR L	None	-	120-150 AC	5.4	297	165	56
BPR L	HSA	1	120-150 AC	5.4	256	181	21
BPR L	HL	1	120-150 AC	5.4	293	192	66
BPR M	None	-	120-150 AC	4.9	553	443	80
BPR M	HSA	1	120-150 AC	4.9	472	443	94
BPR M	HL	1	120-150 AC	4.9	518	468	90

[a]Immersion-compression tests by following methods:

1. Colo.: specimens compacted by kneading compactor with blows at 500 psi; testing strength, AASHO T-165 with 24-hr soaking at 140 F.
2. Utah and Wyo.: specimens compacted by double plunger at 3,000 psi (AASHO T-167); testing strength same as Colo.
3. BPR: specimens compacted as for Utah and Wyo.; testing strength, AASHO T-164 with 24-hr soaking at 140 F.

[b]HSA = concentrated heat stable additive; HL = hydrated lime; PC = portland cement; AF = asbestos fiber; FA = fly ash; LWA = light wt aggregate.

[c]By wt of aggregate.

[d]Fortified at refinery with anti-stripping additive of unknown type and amount.

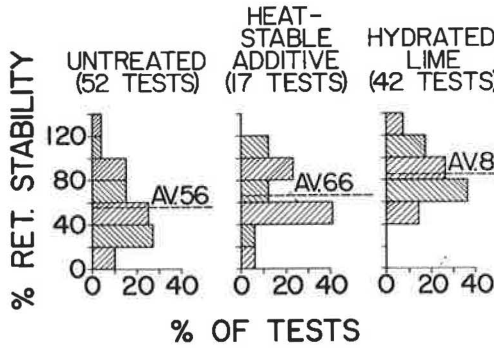

Figure 1. Effect of moisture on bituminous pavement in Rocky Mountain areas.

penetration grade. Retained stabilities in the immersion-compression test are 70 or 75 percent for Interstate and primary roads. Commonly, an additive, either chemical or hydrated lime, must be used to obtain these retained values. Normally, the bituminous mixture is tested without any additive and if the minimum wet-dry stability ratio is not obtained, different commercial chemical anti-stripping additives, hydrated lime, and in some cases, portland cement or other additives are tried. Generally, cement is not as effective as hydrated lime. In some cases, chemical additives give better results than hydrated lime, but in other cases the reverse is true (Table 1, Fig. 1).

EXPERIENCE WITH HYDRATED LIME AND OTHER TREATMENTS

A recent paper presented by Swanson (5) on the use of hydrated lime in asphalt paving mixtures contained rather startling results. For example, with one aggregate the wet stability without any lime was so low that the sample fell apart before it could be tested, but the same mix with 1 percent hydrated lime showed a dry stability of 482 psi and a wet stability of 442 psi for a a wet-dry stability ratio of 92 percent. There was no curing period after adding the hydrated lime. In other cases a curing period of 2 to 5 days was necessary for the lime to react with the aggregate before mixing with asphalt.

On one such job, wet-dry stabilities of 43, 85 and 117 percent, without hydrated lime, with 1 percent hydrated lime and no curing period, and 1 percent hydrated lime with a 48-hr curing period, respectively, were obtained.

The Bureau of Public Roads usually finds that some additive is necessary to obtain a wet-dry stability ratio of 70 percent or more. In some cases a commercial chemical additive is effective, but in others hydrated lime is found to be the more effective additive. Hydrated lime is also considerably more expensive in the proportion normally used (1 to 1.5 percent of weight of aggregate) than chemical additives where only 0.5 to 1.0 percent of the weight of asphalt is necessary. At these proportions, hydrated lime adds about $0.45 a ton to the cost of the mix, whereas the commercial chemical asphalt additive adds about $0.11 a ton. However, if the hydrated lime does the job and the chemical does not, then obviously the additional expenditure is justified. In such case, the proper cost comparison is between the added cost of the hydrated lime and the added cost of some other aggregate not needing the hydrated lime.

Seal coats cost about $0.11/sq yd or on a 3-in. thick mat about $0.70/ton of mix. This is more expensive than either the chemical additive at $0.11/ton of mix or the hydrated lime at $0.45/ton of mix. There is, in addition, no assurance that the seal coat will always prevent the undesirable characteristics of a hydrophilic aggregate from manifesting themselves. Moreover, there are so many uncertainties involved in seal coat construction that the results are somewhat a gamble.

Assuming it has been found that measured by the immersion-compression test, a chemical additive, hydrated lime, or other additive is effective, field behavior should be compared with the laboratory tests. Goldbeck (6) did not find much correlation; however, the Colorado experiment (4) showed good correlation. Whereas it is logical to expect a mix having a high wet-dry stability ratio to be better than one with a low ratio, there are other factors affecting the resistance of the mat to moisture and in some cases they may be of the greater significance and may obscure the action taking place in the immersion-compression test. If the mat is so dense, so well mixed, or so well sealed over, either by compaction or warm weather traffic or by a seal coat, that water does not penetrate it, then stripping, swelling, and loss of stability cannot take place. If it were possible to be sure of keeping water out of the mat and off its surface, there would need be no concern with stripping, raveling, or loss of stability. Obviously there can be no such assurance and, therefore, it is proper to take all possible precautions to prevent mat damage by water.

Whereas there is no exact correlation between immersion-compression values and behavior on the road, there are many cases of stripping, raveling, softening of the mat, and obviously inadequate resistance to the effects of water. The use of chemical additives or hydrated lime will certainly not eliminate these problems because such materials cannot compensate for an inadequately designed or constructed mix, but experience shows they do help.

There are an increasing number of pavements suffering little damage even though not sealed. However, the proportion of lasting as long as 10 yr or even 5 yr without a seal coat is not large. Whether all projects sealed actually need sealing is questionable. The need for sealing is often a matter of personal opinion and some engineers and maintenance men might say a pavement needs sealing, whereas others, with a different background of experience, might not. Because the need for a seal usually develops in winter or spring when it is not possible to do anything about it, it is understandable why some pavements get precautionary seal coats when they might get by without them.

The first winter is usually the critical time because pavements seem to develop increased resistance to the effect of water with time and warm weather traffic. Thus, the loss of an additive's value with time might not matter if its value lasted through the critical early life of the pavement.

LATE-SEASON CONSTRUCTION

As a general rule, pavements placed late in the season require some form of sealing, whereas those placed earlier and subjected to a season of warm-weather traffic before winter arrives are much more apt not to need sealing. Compaction procedures may need to be revised for pavement placed late in the season in order to duplicate by rolling what traffic does to the pavement during warm weather. Logically, this would be an increased amount of rubber-tired rolling while the mat is still warm. Whereas some increased rolling might be performed if necessary to obtain the specified mat density, the specifications do not require any different rolling procedures during cooler parts of the year. Density alone is not the criterion of a good pavement surface to adequately resist the effects of water. Warm-weather rubber-tired traffic tends to knead the pavement surface, resulting in working some asphalt mortar to the surface in much the same manner as working fresh concrete brings concrete mortar to the surface. It is doubtful if it is possible to duplicate the effect of warm-weather traffic on pavements laid late in the season. Asphalt paving material placed on a cold base will cool off quickly in its lower part, whereas the top might still be too warm to permit heavy rolling. However, considerable improvement could be made in cold-weather compaction procedures.

SEAL COATS

Not all seal coats are made necessary by the effect of water on the aggregate-asphalt combination. Many mats become dry and brittle with time, and raveling then starts. Probable causes of this include weathering or hardening of the asphalt or selective absorption of the asphalt into absorptive aggregates.

Seal coats used have generally been of the type using a coarse-graded gravel or crushed rock cover aggregate of about $^3/_8$- to $^1/_2$-in. maximum size applied at about 20 lb/sq yd over an RC cutback used at the rate of about 0.20 gal/sq yd. Most results have been good, but there have also been numerous exceptions when, because of adverse weather, uncontrolled traffic, or a stripping type of cover aggregate, the chips have failed to stick, producing a black, shiny, sticky nonuniform surface (Fig. 2).

Figure 2. Unsatisfactory (excessively rich) seal coat.

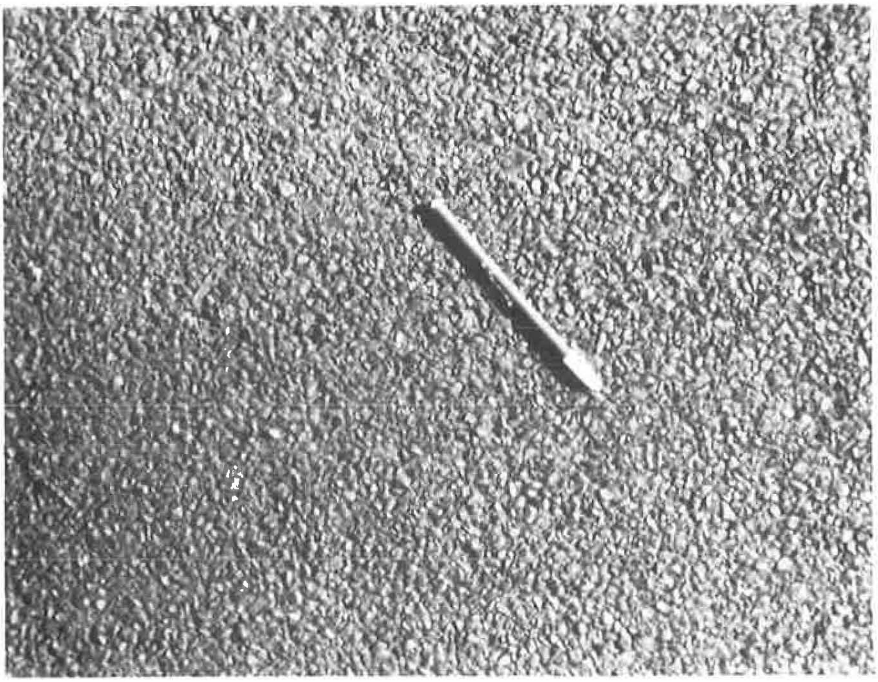

Figure 3. Close-up of plant-mixed seal coat.

In some cases, a sand or sand-gravel cover aggregate has been used, but the results have generally been less uniform and satisfactory than a regular chip seal, although the sand or sand-gravel seals have waterproofed the surface effectively.

A third type of seal frequently used when weather is not suitable for applying a chip or sand seal (as when a mat has been completed late in the construction season) is the so-called fog seal or black seal. A dilute emulsion, SS-1, or light RC or MC cutback is used without cover aggregate. The seal must be applied at a very light rate to prevent a slippery surface. The SS-1 has an advantage in this regard, because it may be diluted to any desired extent with water to provide a complete, yet very thin, cover of asphalt. Rate of application is about 0.02 to 0.05 gal of asphaltic residue per square yard.

A fourth type coming into increased use is the plant-mixed seal in which a semi-open-graded crushed cover aggregate of ⅜-in. maximum size is plant mixed with asphalt cement and spread with a regular paver at about 60 lb/sq yd (Fig. 3). The mix is quite rich (6 to 7 percent asphalt) and must be mixed at a relatively low temperature to permit retention of the required thick film of asphalt. As with the mat itself, an anti-stripping asphalt additive or hydrated lime must often be used to provide adequate resistance to stripping. As in uncoated cover aggregates used in seal coats or surface treatments, the need for an anti-stripping agent is determined by the static-immersion test (AASHO Designation T-182) with 95 percent retention required. Table 2 gives typical results in this test. Typical grading requirements for aggregate in plant-mixed seal are as follows:

Passing ⅜-in. sieve, 100 percent;
Passing No. 4 sieve, 30 to 50 percent;
Passing No. 8 sieve, 15 to 30 percent;
Passing No. 40 sieve, 0 to 10 percent; and
Passing No. 200 sieve, 0 to 3 percent.

This type of seal, if a large quantity is involved or where the contractor is already set up for hot plant-mix work, becomes less expensive than the chip seal and is far superior in appearance and durability.

TABLE 2

STATIC-IMMERSION TEST RESULTS

Utah Project	Type Asphalt	Additive	Stripping (%)
1	SC-3	-	35-90
		1% A	45-50
	RC-4[a]	-	85-90
		1% A	5-10
	120-150	-	70-75
		1% A	0
	RC-4[b]	-	85-90
		1% A	0
2	SC-3	-	75-80
		1% A	25-30
	RC-4[a]	-	60-65
		1% A	5-10
	RC-4[b]	-	15-20
		1% A	0
3	RC-4[a]	-	70-75
		1% A	2-5
	RC-4[b]	-	0
		1% A	0
6	SC-3	-	65-70
		1% A	5-10
	RC-4[a]	-	70-75
		1% A	2
	RC-4[b]	-	2-5
		1% A	0
	120-150[b]	-	5-10
		1% A	0
182-SA-61	MC-3[b]	-	90-95
		1% C	20-25
	RC-4[a]	-	30-35
		1% B	0-2
	120-150[a]	-	5-10
		1% B	0-2
	RS-1[c]	-	10-15
	RS-2[d]		0-2
	RS-2[c]		10-15

[a]Asphalt source. [b]Asphalt source B.
[c]Anionic. [d]Cationic.

Another type of seal receiving increasing acceptance is the so-called slurry seal using a sand, mineral filler and dilute SS-1 emulsion. This seal is inexpensive, but the results have not been as consistently good as the plant-mixed seal described.

When mention is made of seal coating hot plant-mix or asphaltic concrete pavements, surprise is often expressed by engineers from other parts of the country—particularly from farther east—experience has led them to expect a properly constructed pavement to have adequate resistance to water and they, therefore, consider a seal coat to be entirely unnecessary. Even allowing for the possibility that some pavements get seal coated unnecessarily, there still remains a significant difference between the water resistance of pavements in the Rocky Mountain area and those in some other areas. The difference may arise from difference in materials, in climate or something lacking in construction procedures. There are many hydrophilic aggregates, but then probably this is true elsewhere as well.

As an example of what States in the Rocky Mountain area are doing about the problem, Wyoming adopted new design criteria and specifications for plant-mixed surface for the 1962 season. The new specifications require the aggregate to be 100 percent crushed (including fines), provide a grading straddling a maximum density curve (0.45 exponential chart, 7), and require a compacted density of at least 95 percent of standard Marshall (ASTM D-1559). The design criteria requires a minimum wet-dry stability ratio of 70 percent for high-type plant mix, 75 to 85 percent of voids filled with asphalt, and a net void content of 3 to 5 percent in the compacted pavement. Commonly, hydrated lime must be added to obtain the required 70 percent immersion-compression ratio.

Probably because of the high mechanical stability of the 100 percent crushed aggregate, some difficulty has been encountered in obtaining the specified 95 percent plus of Marshall density a very high percentage of the pit material in gravel deposits has had to be wasted. This is not only expensive, but in some areas there is not enough aggregate available to permit such extravagant use. Consequently, the specifications have now been revised to require only 50 percent crushed particles in the plus No. 4 size, and most of the pit fines will now be used. Admittedly the standards have been thereby lowered. In many cases, the pit fines contain undesirable portions of the deposit, but any undesirable characteristics will probably be corrected by the necessity of complying with the 70 percent minimum immersion-compression ratio.

EFFECT OF PI

Tests run on four typical Wyoming aggregates by the Bureau of Public Roads showed that aggregates having PI values of 5 and 6 only gave wet-dry stability ratios of 55 and

60 percent (using Marshall specimens), whereas when these same aggregates had their plastic fines removed by washing and replaced by limestone fines, the ratios jumped to 77 and 86 percent. If the proper minimum wet dry stability ratio is 70 percent using standard 4- by 4-in. double-plunger compacted specimens tested for unconfined compression, the corresponding minimum for Marshall specimens is 75 percent. Washing of aggregates for bituminous pavements has seldom been practiced in this area, but from these tests, it appears to offer one method of improving pavement quality.

The normally accepted limit of 6 PI is proving too high in a great many cases and the general opinion is that the specifications for plant-mix aggregate should require the fines to be nonplastic. Even where specifications do not require nonplastic fines, the 70 percent minimum wet-dry stability ratio requirement makes it necessary to add hydrated lime in most cases where the pit fines have plasticity, thereby resulting in nonplastic fines.

FACTORS INFLUENCING IMMERSION-COMPRESSION RESULTS

The tests generally show better results in the immersion-compression test when heavier asphalts and more asphalt are used. Hydrated lime permits the use of more asphalt and increases dry and wet stabilities. Wet stabilities with lime are often higher than corresponding dry stabilities, possibly because of continued reaction between lime and aggregate fines during the wet soaking period. With some materials, chemical additives are as effective as hydrated lime at increasing wet-dry stability ratios, but tend to reduce the dry stabilities somewhat.

SUMMARY

Bituminous pavements in the Rocky Mountain area have always lacked resistance to the disintegrating effect of water. When road mixes were replaced with plant mixes the problem was reduced, but still existed. Additives—both chemical and hydrated lime—have helped the situation as have closer quality control of the materials and construction procedures. Seal coats have been necessary and have helped to compensate for undesirable stripping and raveling, but the goal is to so design and construct asphaltic concrete pavements that they will not need seal coats. So far there is a way to go before reaching that goal and it is necessary to know just what further changes can be made in design, materials, or in construction procedures to achieve it. Results are improving, but there does not yet seem to be any positive, completely effective solution to the problem. There are indications, however, that considerable improvement in reducing the stripping and raveling problems can be made by close attention to the following details:

1. Determining in advance by laboratory tests, the probable action of the compacted paving mixture in the presence of water. The immersion-compression test is recommended for this purpose.
2. On basis of these tests, either eliminating unsatisfactory aggregates, improving them by screening or washing, or compensating for them by suitable admixtures (chemical anti-stripping additives, hydrated lime, filler or other aggregate sizes to improve gradation).
3. Following good design procedures for the paving mixtures and maintaining close construction control (particularly mixing times, temperatures, and compaction procedures, including more concentrated rolling when the mixture is still warm enough for the rolling to be effective).

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Discussion

ROBERT E. OLSEN, Materials Research Division Bureau of Public Roads.--Mr. Eager has reviewed the problems associated with the use of local aggregates in bituminous construction in some of the Western States and reviewed several practices that are followed in their utilization. The Materials Research Division of the Bureau of Public Roads has been interested in and has followed State practices in bituminous construction in these States for several years and on a number of occasions has cooperated in studies.

A recent study was made to determine the relative quality characteristics of aggregates from four sources in Wyoming. Some of the data collected pertaining to the physical characteristics of these aggregates are given in Table 3; these include aggregate gradation, sand equivalent test results, liquid limit and PI results on the minus No. 40 and minus No. 200 sieve aggregate fractions and hydrometer analysis of the minus No. 200 sieve material.

One phase of the study included the determination of the effect of water on bituminous mixtures prepared with these aggregates. Test specimens were prepared at previously determined optimum asphalt contents and tested by the immersion-compression test for percent retained strength following ASTM Methods D-1074 and D-1075. The immersion period was 4 days at 120 F. In addition, the effects of water on the stability of 50-blow Marshall specimens were determined for mixtures of the same composition and using an immersion period of 1 day at 140 F. The same asphalt, an 85 to 100 penetration grade, was used for all mixtures. The results of these tests and related physical characteristics of the molded specimens are shown in Table 4. A comparison of the data in Tables 3 and 4 shows increasing percentages of retained strength and decreasing percentages of volumetric swell with decreasing values of PI of the material passing the No. 200 sieve and percent clay (material finer than 0.005 mm). To further evaluate these aggregates and to isolate the effect of clay in the bituminous mixture, a series of tests was made with most of the naturally occurring dust, or material passing the No. 200 sieve, removed by washing. Limestone dust was then added in amounts required to bring the total percentage of minus No. 200 sieve material to one half that which the respective aggregates originally contained. The asphalt contents of these mixtures were reduced by 0.5 percent to insure that the air voids would be high enough to allow water to enter the molded specimens and that the asphalt film thickness would not be so great as to pretect the aggregate particles from the effect of water.

The results of tests of mixtures using the washed aggregates are also shown in Table 4. It will be noted that the level of dry stabilities is lower for the washed aggregate mixtures than the unwashed aggregate mixtures in each case; this, however, should be expected with the reduced dust and asphalt contents. The percent retained strength

TABLE 3

PHYSICAL CHARACTERISTICS OF AGGREGATES FROM WYOMING

Type	Passing Sieve (%)									Hydrometer Analysis (%)						Clay[a] (%)	Sand Equiv.	PI	
	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200	Passing No. 200	Finer Than							-No. 40 Fraction	-No. 200 Fraction
											0.50 mm	0.20 mm	0.0005 mm	0.002 mm	0.001 mm				
P	100	90	84	70	55	39	28	17	7.6	100	83	55	36	29	24	2.7	28	5	21
PR	100	88	71	40	25	20	17	12	8.0	100	87	55	31	23	19	2.5	24	6	13
R	100	94	85	63	48	34	25	14	9.4	100	84	45	21	14	11	2.0	31	-	8
GC	100	90	78	53	37	30	21	10	5.9	100	88	56	28	18	14	1.6	46	-	8

[a]In total aggregate calculated from percent finer than 0.005 mm.

TABLE 4

TEST RESULTS OF BITUMINOUS MIXTURES WITH AGGREGATES FROM WYOMING

Type	Unwashed Aggregate										Washed Aggregate							
	A. C. (%)	Compressive Strength[a] (psi)		Retained Strength[a] (%)	Swell[a] (%)	Voids[a] (%)	Stability[b] (lb)		Ret. Stab.[b] (%)	Voids[b] (%)	A. C. (%)	Dust (%)	Limestone Dust Added (%)	Passing No. 200 Sieve (%)	Stability[b] (lb)		Ret. Stab.[b] (%)	Voids (%)
		Dry	Wet				Dry	Wet							Dry	Wet		
P	6.10	246	92	37	4.6	7.8	1,646	900	55	4.3	5.60	1.8	2.0	3.8	1,192	912	77	6.3
PR	5.25	246	105	43	3.4	6.7	1,525	910	60	4.3	4.75	1.0	3.0	4.0	1,252	1,074	86	4.8
R	5.75	289	192	66	1.6	6.5	2,044	1,570	77	3.9	5.25	1.3	3.4	4.7	1,560	1,508	97	4.3
GC	4.50	249	181	73	1.2	6.9	1,678	1,350	80	3.7	4.00	0.8	2.2	3.0	1,236	1,249	101	4.8

[a]Specimens 4 by 4 in. [b]Marshall specimens.

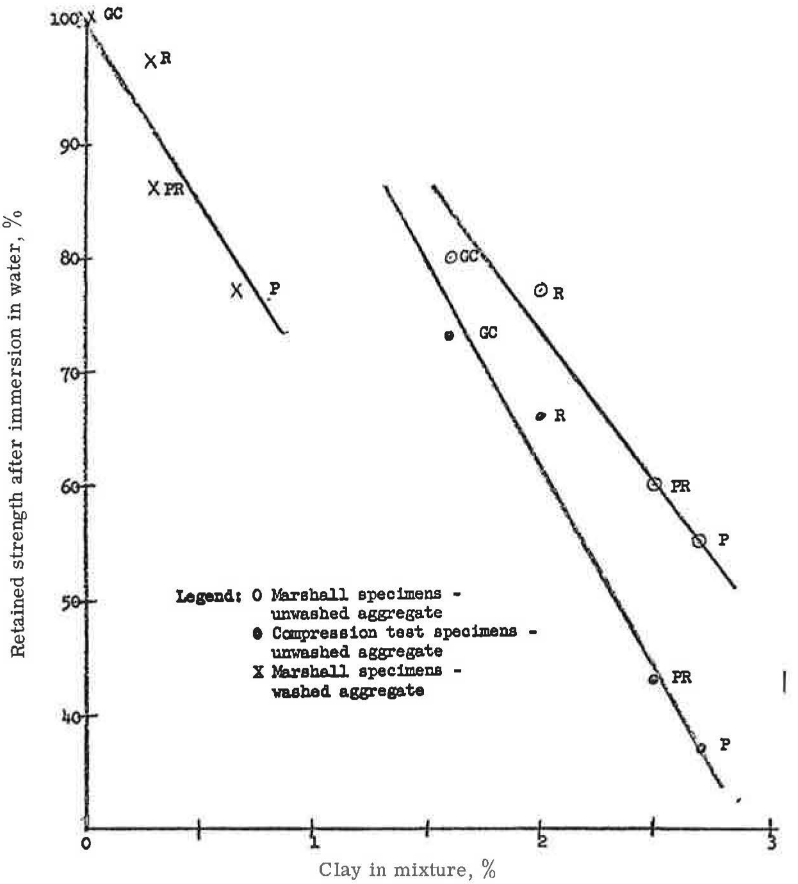

Figure 4. Relationship of percent clay in bituminous mixtures and percent retained strength.

of each mixture, however, is from 25 to 50 percent higher than the comparable mixtures using the unwashed aggregates. This supports the statement that the washing of aggregate may be a logical step toward the upgrading of local aggregates.

Figure 4 shows graphically the relationship of percent retained strength of molded specimens to percent clay in the respective aggregates. The percent clay in the washed aggregates is based on the percent passing the No. 200 sieve after washing and the percentage of material finer than 0.005 mm as determined by the hydrometer analysis of the original fines. The percent clay in the washed aggregates is, therefore, not absolutely correct.

Application of Statistical Mechanics to Analysis Of Degradation of Aggregates

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The problems encountered in the formulation of a theory of degradation of aggregates are indicated and a general equation based on principles of statistical mechanics is developed. The equation is based on the assumption that in a heterogeneous mass (such as aggregate) cracks, flaws and planes of weakness are normally distributed in each fraction. This equation relates the reduction in size to the original size and surface energy of the aggregate and explains the limitations of existing comminution hypotheses such as those of Rittinger, Kick and Bond.

The experimental results obtained from the compaction of one-sized and combinations of different sizes of aggregate in a gyratory machine show that the kind of aggregate, gradation, maximum size, presence or absence of asphalt, and degree of compaction do not affect the pattern of degradation of each fraction. It is found that these variables will only alter the magnitude of the breakage energy received by each fraction during the compaction process.

•ONE OF the important objectives of highway engineering is the prediction of the performance of highway materials during construction and the life of the highway. Aggregate is one of the most commonly used road-building materials. A review of the literature concerning aggregate qualities shows that many methods have been used to determine the suitability of aggregates for road-building purposes. This can be traced as far back as 1819 when John MacAdam stated that the workmen determined the proper size of stone by measuring them with their fists (1).

Testing of road materials, including aggregates, started in the United States about 1890 (2) and has developed steadily since that time. Much work has been done to relate the results of mechanical tests on road stones to the life and performance of the roads in which they were used as aggregates (3).

In bituminous roads, forces due to traffic tend to cause degradation. Macnaughton (4) states that one of the features which has to be considered in the design of bituminous mixtures is the matter of degradation of aggregates during the process of compaction. He further states that this degradation is due to considerable pressure developed at points of contact between particles and to continual shifting and rearrangement of particles as they gradually attempt to occupy the least possible space. Workers in the Road Research Laboratory of England (5) have substantiated the orientation of particles in bituminous roads by stating, "The orientation of the aggregate in a section of road carpet is largely determined by the boundaries. In the carpet most of the aggregate particles are arranged with their long axis horizontal." Increase of pressure and reorientation of particles produce a grinding effect that tends to round off corners and

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edges, thus permitting the particles to fit more closely together. This process may proceed so far that aggregate void space is materially reduced and the stability of the mixture is impaired by the presence of an excess of liquid (4).

Smith (7) states: "... some mixes of adequate internal friction and cohesion at the time of construction became over-lubricated due to densification under traffic. As the result of this densification, the stability properties (internal friction in particular) of the mix as measured by the triaxial test were greatly reduced." Pauls and Carpenter (8) have concluded that the principal cause of degradation is the rolling operation, and excessive degradation of the aggregate in bituminous concrete under the roller may cause raveling, excessive absorption of water, and progressive deterioration. Benson (9) considers that the relocation of particles occurs as soon as the friction between the particles is overcome, and bituminous materials assist in the action by acting as a friction reducer. He states, "... within a bitumen-aggregate structure, one of the main roles of bitumen is that it permits a re-establishment of the original resistance to deformation when the bitumen-aggregate structure is deformed by excessive stress and is then compacted by further action of traffic." H. L. Day of the Idaho Department of Highways reports (10) that construction records in that State indicate a pavement built with a shale aggregate failed before construction was completed and had to be reconstructed before being put into service.

Compaction is an energy-consuming process resulting from the application of forces to the mixture. The mixture withstands these forces in many ways, such as by interlock, by frictional resistance, and by viscous or flow resistance. When applied forces have a component in any direction less than that required to cause failure but greater than the resistance of the mat, the material moves and shifts 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 that 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 the basic properties of stability and durability of the bituminous mixture.

ANALYSIS OF MECHANISM OF DEGRADATION

Most civil engineers concerned with degradation of aggregate believe that the process involves energy and that particles reduce in size by virtue of work done by the forces that develop at contact points (11). However, a careful review of the existing literature seems to indicate that no one has tried to analyze the problem using either the principles of energy consumption or fracture or breakage of solid pieces, and no attempt has been made to evaluate the factors affecting this process. It is interesting to note, however, that a vast amount of information predicated on this principle is available in the mining and metallurgical fields, especially in literature dealing with ore treatment processes where crushing and grinding is frequently the major item of cost.

Most comminution theories are concerned with the relationship between the cause and effect of particle size reduction. Of the many theories in the mining and metallurgical fields suggested for the relation between input energy (cause) and reduction in particle size of materials (effect) those of Rittinger, Kick, and Bond are of special interest (12).

Rittinger in 1867 postulated that the energy required for size reduction of a solid is proportional to the new surface area created during the size reduction. Kick in 1885 suggested that equivalent amounts of energy should result in equivalent geometrical changes in the size of the pieces of solid. Holmes (12) notes that Rittinger's theory is related to the energy required for the rupture of chemical and physical bonds, whereas Kick's theory refers to the energy required to deform the particle to its elastic limit.

A more recent theory of comminution, which can be considered a compromise between Kick's and Rittinger's theories, was provided by Bond (13) in 1952. Bond's theory states: "The work, useful in breakage, which has been applied to a weight of homogeneous broken material is inversely proportional to the square root of the diameter of the product particles."

These three theories were represented in a single, simple empirical proposition by Charles (14) in 1957:

$$dE = -c\, dx/x^n \tag{1}$$

in which

dE = infinitesimal energy change,
c = a constant,
dx = infinitesimal size change,
x = particle size, and
n = a constant.

This equation states that the energy required for a small change in the size of a particle is inversely proportional to the particle size to some power n and directly proportional to the size change. Charles further proves that when $n = 2$, Eq. 1 yields Rittinger's hypothesis. Kick's concept is obtained when $n = 1$, and the value, $n = 1.5$, results in Bond's theory. There are many experimental results supporting the validity of each of the three hypotheses (13, 15, 16). However, Holmes (12) notes that because the hypotheses do not consider the characteristics of the materials, none of them can be accepted completely. He interprets the experimental support of the hypotheses as being due to the limited range of the n value of Eq. 1.

In order to develop any general equation for size reduction, the characteristics of the rocks must be considered. Real aggregates are generally composed of nonhomogeneous rock fragments with internal weaknesses, flaws, cracks, etc., within them. However, as the particle sizes decrease, the influence of these weaknesses diminishes and more energy is required to produce a unit surface area.

In coarse pieces breakage occurs due to rupture of Van der Waals' bonds between crystals, whereas in small particles (of sizes on the order of crystalline dimensions) fractures must occur within crystals and, consequently, sufficient energy must be available to overcome valence bonds. In the latter case the energy required to produce a unit surface area is inversely proportional to the diameter of the particle to some unknown power.

In the development that follows it will be assumed that distribution of weaknesses in a group of particles of relatively uniform size is normal. The same assumption will be made for the energy required for the breakage of each particle. That is, the mean value of the energy distribution will be considered as a representative value for a particular particle size.

When an aggregation of particles of one size is subjected to external energy, a part of the energy is used to produce particles of smaller sizes. An increase in the input energy will result in an increase of the number of particles of smaller sizes. Ultimately this process can be considered to produce particles of almost zero size; that is, any further increase in energy will cause chemical rather than physical changes. Symbolically, this can be stated as: a particle of size d_i requires an input energy of w_i to break into particles of zero size, and w_i increases as d_i increases. On the basis of this definition, N_i particles of d_i size will consume $N_i w_i$ energy to change into particles of zero size and $\sum_{i=0}^{n} N_i w_i = U$ will be the total breaking energy that an aggregation of particles of different sizes require to change into particles of zero size. Considering M the total mass of the material and m_i the mass of each fraction, $\sum_{i=0}^{n} m_i = M$:

we shall define $p_i = (m_i / M)\, 100$ as the percentage of material of size d_i, in which $\sum_{i=0}^{n} p_i = 100$. For simplicity, assuming spherical particles

$$N_i = \frac{m_i}{\frac{\pi}{6} d_i^3 \rho} \tag{2}$$

in which $\frac{\pi}{6} d_i^3$ is the volume of the particle and ρ is the density of the material. For nonspherical particles the volume can be considered Ad_i^3, in which A is a shape factor. The total breaking energy now can be written

$$U = \sum N_i w_i = \sum \frac{m_i}{\frac{\pi}{6} \rho d_i^3} w_i \tag{3}$$

If the value of w_i is used for unit surface area of the particle rather than the particle as a whole, the relationship between the internal energy, w_i, of a particle of size d_i and the energy per unit surface area, E_i, is

$$w_i = \pi d_i^2 E_i \tag{4}$$

which, when substituted into Eq. 3 yields

$$\frac{6}{\rho M} \sum p_i \frac{E_i}{d_i} = U \tag{5}$$

The point of interest to those concerned with degradation is to predict the gradation of degraded aggregate after certain energy consumption. It is highly desirable to know what would be the values of p_i for various sizes, that is, the most probable distribution of values of p_i for each size d_i when p_i may take any value from zero to 100 percent. The number of combinations of p_i is given by the permutation formula

$$W = \frac{P!}{\pi p_i!} \tag{6}$$

in which P is total percentage of material smaller than the largest size considered, which is generally 100 percent, and $\pi p_i!$ is the product of the values of $p_i!$, W is a term similar to that used in statistical mechanics for "thermodynamic probability" giving the number of microstates corresponding to a given macrostate. In statistical mechanics, the macrostate corresponds to the specification of how many molecules are in each energy level and the microstate to that of which molecules are in which energy level.

In the degradation process, W can be considered the degradation probability, the macrostate the number of particles in each fraction and the microstate which particles are in which fraction. For this purpose the most probable combination of p_i's which in turn will correspond to a maximum W is sought.

After application of an input energy, Eq. 5 is equal to a constant, and due to conservation of mass $\Sigma p_i = P_o$ = constant. Differentiating to obtain the maximum W results in

$$p_i = \alpha \exp \beta E_i / d_i \tag{7}$$

in which α and β are constants. From $\Sigma p_i = P_o$ is obtained

$$\alpha = \frac{P_o \beta}{(\exp \beta E_o / D_o) - 1} \tag{8}$$

in which D_o is maximum size of particles. Substituting α in Eq. 7 for p_i yields

$$p_i = P_o \beta \frac{\exp \beta E_i / d_i}{(\exp \beta E_o / D_o) - 1} \tag{9}$$

Consequently, the total percent passing any size may be expressed as

$$p_i = \sum_{i=0}^{d_i} p_i = \frac{P_o \beta}{(\exp \beta E_o / D_o) - 1} \int_0^{E_i / d_i} (\exp \beta E_i / d_i) \, d\,(E_i / d_i) \tag{10a}$$

or

$$p_i = \frac{P_o \left[(\exp \beta E_i / d_i) - 1\right]}{(\exp \beta E_o / D_o) - 1} \tag{10b}$$

Finally, if the values of $\exp \beta E_i / d_i$ and $\exp \beta E_o / D_o$ are much larger than unity and - 1 may be ignored, the percent of material p_i finer than size d_i after the application of a certain amount of energy is

$$p_i = P_o \exp - \beta (E_o / D_o - E_i / d_i) \tag{11}$$

RELATION OF EXPERIMENT AND THEORY

Materials, Equipment and Procedure

To check the validity of Eq. 11, a series of tests was performed on three kinds of mineral aggregates. The details of quality and specific characteristics of the aggregates along with the tests performed on them are discussed elsewhere (18, 19). Tables 1 and 2 summarize data on origin, specific gravity, Los Angeles value, compressive strength, and petrographic analysis results for the materials used. An 85 to 100 penetration asphalt cement was used in the study.

Compaction was accomplished by a gyratory testing machine, which incorporates horizontal forces and applies shear to the specimen throughout its depth. Because the materials in the field are under similar loading conditions, the specimens produced in this machine should have a similar structure to that produced from the same mixture

TABLE 1

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

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

[a]Each value is the average of 3 tests. [b]Rate of loading, 0.025 ipm.
[c]According to ASTM Method C-131.

TABLE 2

PETROGRAPHIC ANALYSIS

Item	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
Av. 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 of limonite and hematite.	Total % by vol, 1 limonite, hematite.	Hematite as coatings and finely disseminated grains, sericite in seams and disseminated throughout.
Secondary cementation	Absent	Unobservable	
Voids (%)	6.0	0.7	0.5
Nature of grain boundaries	Loose interlocking.	Good interlocking.	Rock and grains are both highly fractured (cataclastic structure); all quartz grains display a prominent wavy extinction, indicating a highly stressed rock.
Fracturing and cracking	Low	Insignificant	
Particle orientation	Random, sometimes lineation due to decomposition.	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 to very fine mesh.	Recemented granulated matrix.

in the field. With this machine the compactive effort could be changed in magnitude of load and repetition of load.

Results

The results of degradation produced on the aggregates tested are shown by gradation curves of percent smaller than certain sizes on log-log plots. All tests were performed using a constant angle of gyration of 1° with a fixed roller.

Degradation of One-Sized Aggregate. —The results of degradation of one-sized aggregate are presented in Table 3 for 12 specimens. Specimens containing 1,000 gm of one-sized aggregate of ½ to ⅜ in., ⅜ in. to No. 3, No. 3 to No. 4, and No. 4 to No. 6 of each of the three aggregates, dolomite, limestone and quartzite, were compacted in the gyratory compactor under 200 psi ram pressure and 100 rev.

Figures 1 through 3 show the results of sieve analysis on the degraded aggregates. Figures 4 through 6 show similar results on the three kinds of aggregate under different gyratory compactive efforts. The compactive effort for these specimens was varied by number of repetitions of load shown on each curve. These results are given in Table 4.

Degradation of Combination of Aggregate. —To investigate how the theory holds when aggregates of different sizes are mixed and subjected to a certain gyratory compactive effort, it was necessary to dye the pieces of each fraction a different color so that after compaction they could be separated by their colors.

The results of such tests are given in Table 5 for limestone aggregate when four different sizes were combined equally, compacted by gyration and separated according to their colors. Table 6 contains such results when the aggregate, before gyratory compaction, was mixed with 4 percent asphalt. Figures 7 and 8 show typical results of degradation of each fraction for specimens without and with 4 percent asphalt, respectively.

Discussion of Results and Conclusions

Most investigators have suggested that in any size reduction process the gradation of degraded aggregates can be expressed mathematically by a power function such as

TABLE 3

RESULTS OF GYRATORY TESTS OF VARIOUS ONE-SIZED AGGREGATES[a]

Original Grading	Passing Sieve (%)									
	½ In.	⅜ In.	No. 3	No. 4	No. 6	No. 8	No. 16	No. 50	No. 100	No. 200
(a) Dolomite										
½ - ⅜ in.	100.0	59.8	37.3	30.6	25.2	21.3	14.2	7.2	5.4	3.8
⅜ in. - No. 3	-	100.0	53.6	37.4	29.6	24.5	16.4	8.1	6.0	4.1
No. 3 - No. 4	-	-	100.0	48.5	32.5	25.8	16.7	8.4	6.1	4.5
No. 4 - No. 6	-	-	-	100.0	46.5	31.0	18.7	9.0	6.8	5.0
(b) Limestone										
½ - ⅜ in.	100.0	55.3	32.0	24.9	20.2	16.5	10.7	4.7	2.9	1.8
⅜ in. - No. 3	-	100.0	58.4	34.1	25.7	19.8	12.1	4.8	3.1	2.0
No. 3 - No. 4	-	-	100.0	54.3	33.7	24.7	14.7	5.8	3.6	2.2
No. 4 - No. 6	-	-	-	100.0	53.6	32.3	17.0	6.2	3.8	2.4
(c) Quartzite										
½ - ⅜ in.	100.0	48.6	23.2	17.9	14.0	11.3	7.0	3.1	1.8	1.1
⅜ in. - No. 3	-	100.0	43.8	26.6	19.2	14.8	8.8	3.5	2.1	1.3
No. 3 - No. 4	-	-	100.0	37.0	19.3	14.5	8.3	3.4	2.2	1.5
No. 4 - No. 6	-	-	-	100.0	38.1	20.8	10.6	3.7	2.4	1.6

[a]Two hundred psi, 100 rev. 0% asphalt.

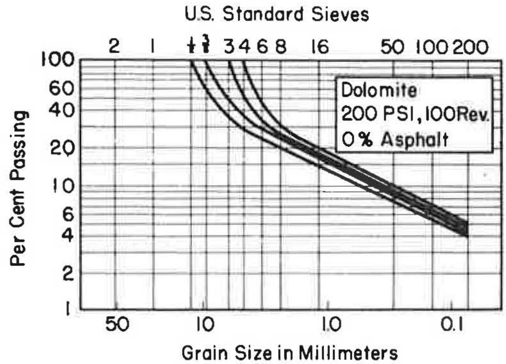

Figure 1. Sieve analysis of gyrated dolomite aggregate.

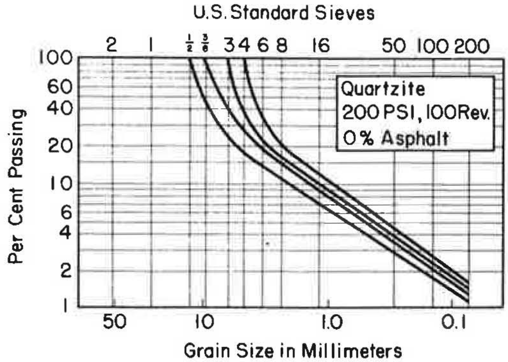

Figure 2. Sieve analysis of gyrated quartzite aggregate.

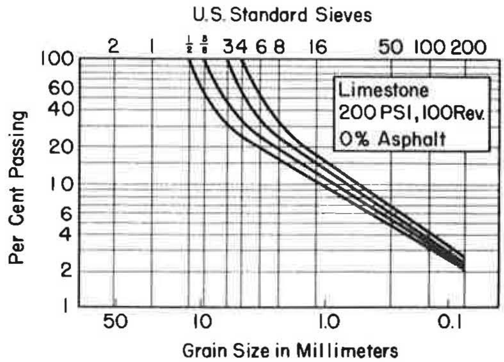

Figure 3. Sieve analysis of gyrated limestone aggregate.

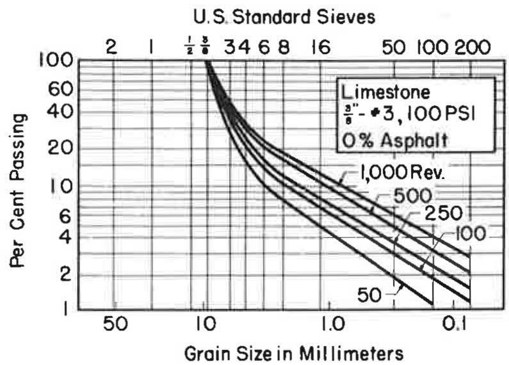

Figure 4. Sieve analysis of gyrated one-sized limestone aggregate.

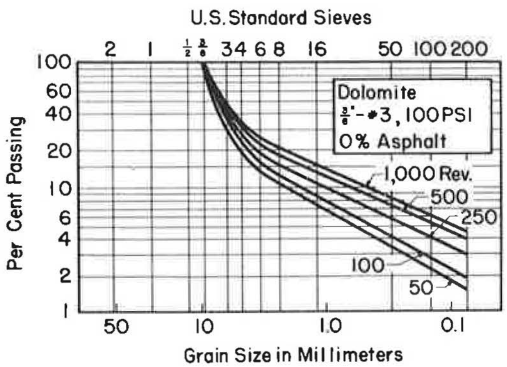

Figure 5. Sieve analysis of gyrated one-sized dolomite aggregate.

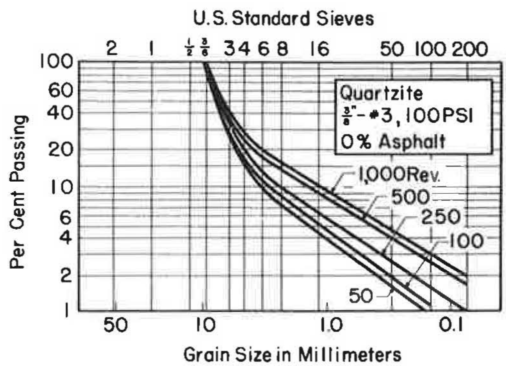

Figure 6. Sieve analsyis of gyrated one-sized quartzite aggregate.

TABLE 4

RESULTS OF GYRATORY TESTS OF ONE-SIZED AGGREGATES[a]

Revolutions (no.)	Passing Sieve (%)								
	3/8 In.	No. 3	No. 4	No. 6	No. 8	No. 16	No. 50	No. 100	No. 200
				(a) Dolomite					
50	100.0	34.3	19.5	14.5	11.1	6.6	3.2	2.4	1.7
100	100.0	42.9	22.0	16.0	12.8	7.3	3.6	2.7	2.0
250	100.0	45.6	25.8	20.3	15.5	10.0	5.5	4.1	2.8
500	100.0	48.2	30.7	23.0	18.5	14.0	7.3	5.5	3.9
1,000	100.0	50.0	32.1	25.5	20.6	15.6	8.3	6.1	4.3
				(b) Limestone					
50	100.0	29.7	16.8	11.0	8.5	4.6	1.8	1.2	0.8
100	100.0	36.7	21.2	14.8	11.6	6.6	2.7	1.8	1.3
250	100.0	38.9	23.3	16.6	13.4	8.2	3.5	2.3	1.5
500	100.0	41.9	27.4	20.1	16.6	10.7	4.7	3.1	2.1
1,000	100.0	47.3	30.5	23.1	19.8	13.4	6.2	4.1	2.7
				(c) Quartzite					
50	100.0	28.8	15.2	10.4	7.6	4.1	1.5	0.9	0.5
100	100.0	31.6	17.1	11.9	8.8	4.9	1.9	1.1	0.7
250	100.0	35.7	20.0	14.3	10.9	6.4	2.6	1.6	1.0
500	100.0	39.8	23.8	17.8	14.0	9.0	4.1	2.4	1.5
1,000	100.0	43.7	27.5	20.7	16.5	10.8	4.9	3.0	1.8

[a]Grading 3/8 in. to No. 3, 100 psi, 0% asphalt.

TABLE 5

RESULTS OF SIEVE ANALYSIS OF COLORED AGGREGATES[a]

Aggregate Size and Color	Passing Sieve (%)										Total Wt (gm)
	½ In.	3/8 In.	No. 3	No. 4	No. 6	No. 8	No. 16	No. 50	No. 100	No. 200	
				(a) Compaction: 100 Psi, 30 Rev.							
Violet, ½-3/8 in.	100.0	25.5	11.6	8.2	5.6	4.0	1.9	0.9	-	-	250.0
Red, 3/8 in. - No. 3	-	100.0	24.3	10.0	7.0	5.2	3.2	1.6	-	-	251.0
Green, No. 3 - No. 4	-	-	100.0	31.1	15.0	10.4	5.3	2.0	-	-	251.0
Natural, No. 4 - No. 6	-	-	-	100.0	48.2	23.7	9.6	2.2	-	-	251.0
Total	100.0	81.4	59.0	37.3	18.4	10.8	5.0	1.6	1.0	0.7	1,003.0
				(b) Compaction: 100 Psi, 100 Rev.							
Violet, ½-3/8 in.	100.0	27.7	13.2	10.0	7.0	5.2	3.2	2.0	-	-	251.5
Red, 3/8 in. - No. 3	-	100.0	32.4	14.7	10.3	8.1	5.0	2.5	-	-	251.5
Green, No. 3 - No. 4	-	-	100.0	40.4	22.1	15.5	8.0	3.0	-	-	251.5
Natural, No. 4 - No. 6	-	-	-	100.0	59.8	32.4	15.6	4.0	-	-	251.5
Total	100.0	83.3	61.0	40.8	24.5	15.2	7.9	2.8	1.8	1.1	1,006.0
				(c) Compaction: 200 Psi, 30 Rev.							
Violet, ½-3/8 in.	100.0	44.0	19.4	14.0	10.8	8.6	5.4	2.9	-	-	250.0
Red, 3/8 in. - No. 3	-	100.0	45.6	20.5	13.9	10.9	6.1	3.5	-	-	251.0
Green, No. 3 - No. 4	-	-	100.0	43.0	24.5	16.9	9.5	4.6	-	-	251.0
Natural, No. 4 - No. 6	-	-	-	100.0	69.1	39.8	17.3	5.9	-	-	251.0
Total	100.0	86.0	66.8	44.7	29.6	19.1	9.6	3.3	2.1	1.3	1,003.0
				(d) Compaction: 200 Psi, 100 Rev.							
Violet, ½-3/8 in.	100.0	52.2	23.6	16.6	12.8	10.2	7.1	4.6	-	-	249.8
Red, 3/8 in. - No. 3	-	100.0	49.4	22.2	16.4	12.6	8.2	5.2	-	-	249.8
Green, No. 3 - No. 4	-	-	100.0	49.4	28.4	20.8	11.9	6.9	-	-	250.0
Natural, No. 4 - No. 6	-	-	-	100.0	77.2	48.8	24.0	7.8	-	-	250.0
Total	100.0	88.1	68.3	47.1	33.7	23.1	12.0	4.9	3.1	1.9	999.5

[a]Grading 0, 0% asphalt.

TABLE 6

RESULTS OF SIEVE ANALYSIS OF COLORED AGGREGATES[a]

Aggregate Size and Color	Passing Sieve (%)											Total Wt (gm)
	1/2 In.	3/8 In.	No. 3	No. 4	No. 6	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	
(a) Compaction: 100 Psi, 30 Rev.												
Violet, 1/2-3/8 in.	100.0	19.6	6.2	4.4	3.0	2.2	1.1	0.5	-	-	-	250.0
Red, 3/8 in. - No. 3	-	100.0	25.4	8.4	4.8	3.4	1.5	0.7	-	-	-	250.0
Green, No. 3 - No. 4	-	-	100.0	28.4	11.6	7.2	3.5	2.0	-	-	-	250.0
Natural, No. 4 - No. 6	-	-	-	100.0	49.4	24.6	10.5	5.8	-	-	-	250.0
Total	100.0	79.9	57.9	35.3	17.2	9.4	4.1	2.2	1.3	0.8	0.5	1,000.0
(b) Compaction: 100 Psi, 100 Rev.												
Violet, 1/2-3/8 in.	100.0	25.0	11.0	8.0	5.0	3.0	1.8	1.0	-	-	-	250.0
Red, 3/8 in. - No. 3	-	100.0	29.6	11.0	7.0	4.8	3.0	2.0	-	-	-	250.0
Green, No. 3 - No. 4	-	-	100.0	36.2	16.2	10.8	5.7	4.3	-	-	-	250.0
Natural, No. 4 - No. 6	-	-	-	100.0	55.2	30.8	14.9	9.1	-	-	-	250.0
Total	100.0	79.4	58.9	38.0	20.2	11.9	5.7	3.4	2.1	1.5	1.0	1,000.0
(c) Compaction: 200 Psi, 30 Rev.												
Violet, 1/2 - 3/8 in.	100.0	30.5	14.9	10.1	7.9	5.7	2.9	1.8	-	-	-	250.0
Red, 3/8 in. - No. 3	-	100.0	36.5	14.5	10.3	6.9	3.8	2.8	-	-	-	250.0
Green, No. 3 - No. 4	-	-	100.0	45.6	25.4	18.0	9.2	6.9	-	-	-	250.0
Natural, No. 4 - No. 6	-	-	-	100.0	60.6	35.2	20.2	13.0	-	-	-	250.0
Total	100.0	84.1	62.9	42.6	28.1	18.0	9.1	5.4	3.4	2.2	1.5	1,000.0
(d) Compaction: 200 Psi, 100 Rev.												
Violet, 1/2-3/8 in.	100.0	34.0	17.0	12.0	8.6	7.1	4.1	2.6	-	-	-	250.0
Red, 3/8 in. - No. 3	-	100.0	43.0	20.2	13.6	10.0	5.8	3.5	-	-	-	250.0
Green, No. 3 - No. 4	-	-	100.0	48.0	29.2	21.2	12.6	9.2	-	-	-	250.0
Natural, No. 4- No. 6	-	-	-	100.0	65.4	39.7	23.5	17.0	-	-	-	250.0
Total	100.0	83.0	64.5	44.8	29.2	19.5	10.5	6.3	3.9	2.5	1.6	1,000.0

[a]Grading 0, 4% asphalt.

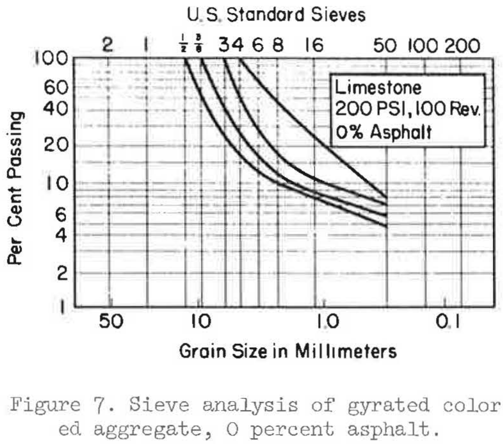

Figure 7. Sieve analysis of gyrated colored aggregate, 0 percent asphalt.

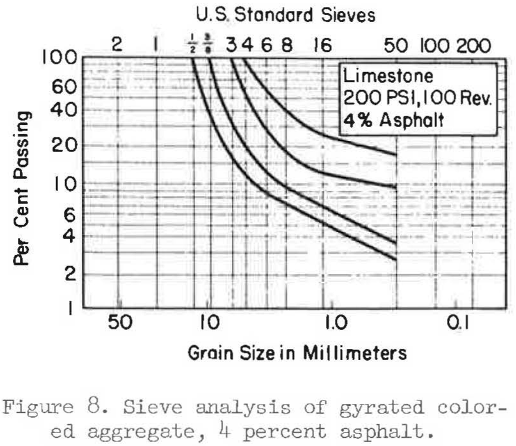

Figure 8. Sieve analysis of gyrated colored aggregate, 4 percent asphalt.

$$P_i = P_o\left(\frac{d_i}{D_o}\right)^n \tag{12}$$

in which

P_i = total percent of material smaller than size d_i,
P_o = total percent of material smaller than size D_o,
D_o = maximum size of degraded aggregate or the size such that
P_o = 80 percent, and
n = a constant whose value depends on the crushing device and type of mineral.

If this type of equation holds for any size reduction process, the plot of total percent passing vs the size of the aggregate on a log-log scale should be a straight line. Deviations from straight lines, in the coarse aggregate range of the plots, are reported by almost all investigators, showing that the value of n is not independent of particle size. The results presented in this study show that deviation from a straight line depends not only on the size of the particles but also on the magnitude of compactive energy applied to the specimen (Figs. 4, 5, and 6).

Figures 1, 2, and 3 show that the slope of the straight portion of the curves for each type of aggregate is almost constant. This indicates, in the fine-particle range where the influence of the particle size is negligible, that size distribution can be predicted by equations similar to Eq. 12. However, a comparison of the three figures shows that this slope varies from one kind of aggregate to another. This is true because the energy required to create fracture in any kind of rock depends on chemical and mineralogical composition of the rock.

The foregoing variables that are not included in the empirical expressions developed by Rittinger, Kick, Bond and others have limited the application of these expressions. However, the size of the aggregate, its chemical and mineralogical composition, and the magnitude of the compactive effort are incorporated in the expression developed in this study through the term $-\beta(E_o/D_o - E_i/d_i)$. Using the relationship

$$(D_o/d_i)^{-n} = \exp - n \ln (D_o/d_i) \tag{13}$$

in which ln is the natural logarithm, Eq. 12 becomes

$$P_i = P_o \exp - n \ln (D_o/d_i) \tag{14}$$

Equating Eqs. 11 and 12 yields

$$\exp - \beta (E_o/D_o - E_i/d_i) = \exp - n (\ln D_o - \ln d_i) \tag{15a}$$

or

$$\beta (E_o/D_o - E_i/d_i) = n \left(\ln \frac{1}{d_i} - \ln \frac{1}{D_o}\right) \tag{15b}$$

This relationship shows that the n value suggested by other investigators depends on E_o and E_i which are dependent on size and kind of aggregate and on compactive effort. This can be shown further by considering $n = \beta\gamma$ where Eq. 15b stipulates

$$E_o/D_o - E_i/d_i = \ln\left(\frac{1}{d_i}\right)^{\gamma} - \ln\left(\frac{1}{D_o}\right)^{\gamma} \tag{16}$$

This shows clearly that γ is a function of E_o, E_i, D_o, and d_i. Whereas E_o and D_o are constants for any size and type of aggregate, γ must be a function of breakage energy E_i and particle size d_i.

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Constructing Longitudinal Joints In Hot-Mix Asphalt Pavements

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This paper presents results of density and tensile strength test of samples cut from longitudinal joints in hot-mix asphaltic pavement. An unanticipated finding is that in semi-hot and cold joints there is a low-density zone at the joint in the lane paved first and a high-density zone at the joint in the lane paved subsequently. These zones are not present in hot joints made with pavers operating in echelon. These low- and high-density zones may well be the basic problem in constructing durable longitudinal joints in asphalt pavements.

•JOINTS are often the weakest portion of a bituminous concrete surfacing and under unfavorable conditions of exposure and/or construction, visible defects may occur first at the joints. These defects may allow the ingress of water into the pavement leading to further disintegration (Fig. 1). This paper presents density and tensile strength of longitudinal joints made by several methods.

A search of the literature revealed no pertinent information on the subject. Therefore, standard specifications of the various States were used as references on joint construction techniques.

Hot Joints

These are produced with pavers operating in echelon spaced close enough together so that the lane placed first does not cool significantly before the second lane is placed. There is no special treatment of the face of the joint placed first. Rolling techniques can be "pinched" or "overlapped".

Semi-hot Joints

These are produced when a restriction is placed on the distance a paver may proceed before setting back and bringing up the adjacent lane to match the first lane. Generally the material in the first lane cools to approximately 120 to 140 F before the adjacent lane is placed.

First Lane.—The edge of the first lane, which will be a face of the joint, may be compacted during rolling of the first lane prior to placing the second lane, or a strip approximately 6 in. wide may be left uncompacted until the subsequent lane is placed and rolled. Usually, there is no treatment of the edge of the first-laid lane, although in some instances, a tack of cut-back or emulsion may be required.

Subsequent Lane.--In placing the subsequent lane, the paver may be operated to butt the joint or it may overlap the first lane 2 to 4 in. The overlapped material may be scooped up with a shovel to a neat line at the joint, swept back on the hot lane, or pushed

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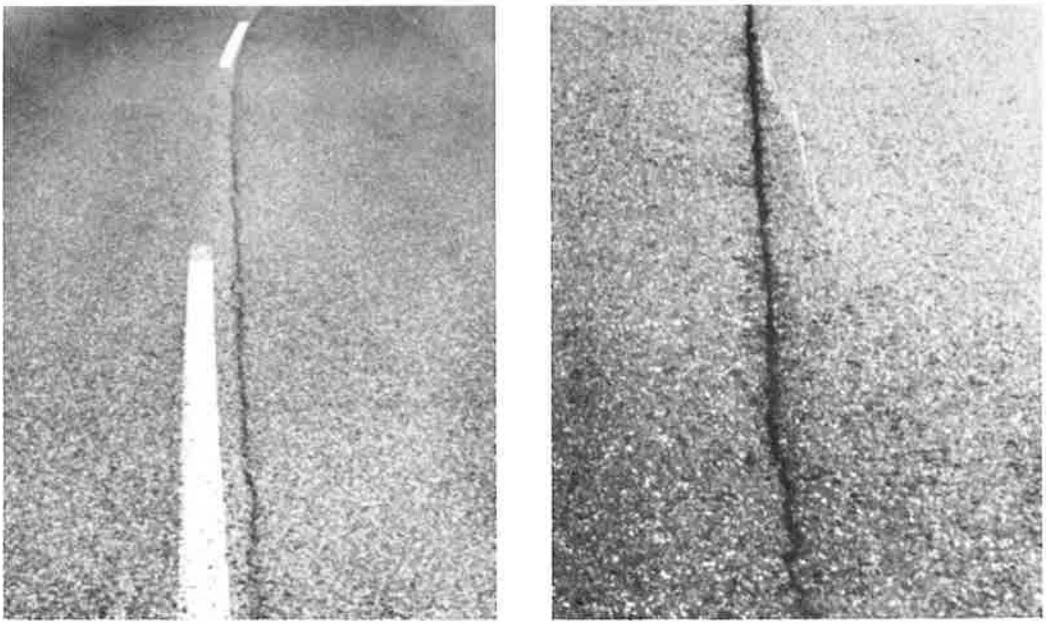
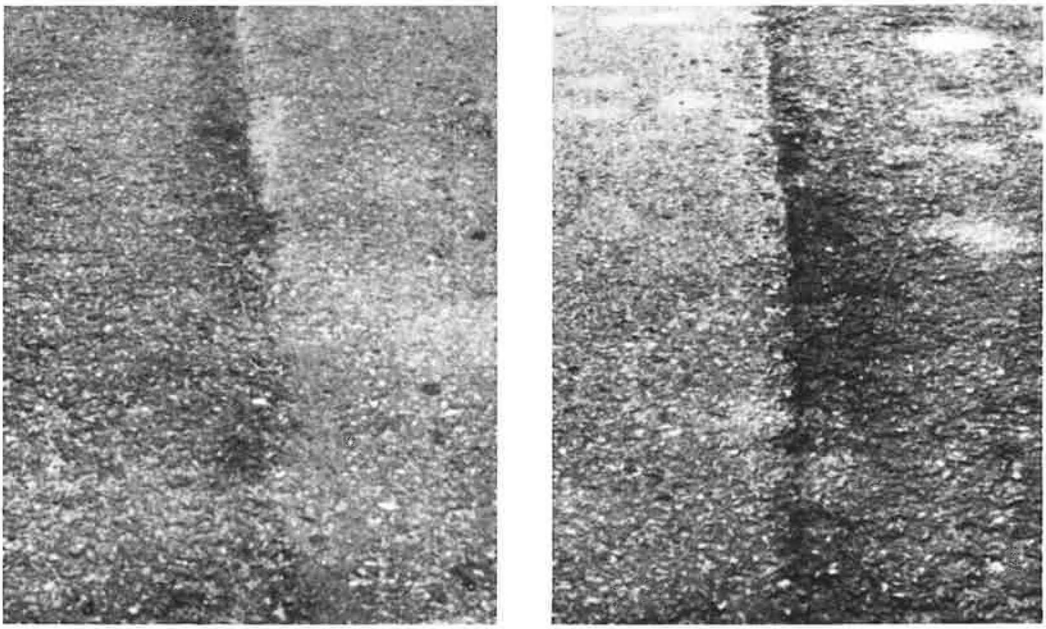


(a)



(b)

Figure 1. Visible defects at joints: (a) center line cracking, and (b) center joint raveling.

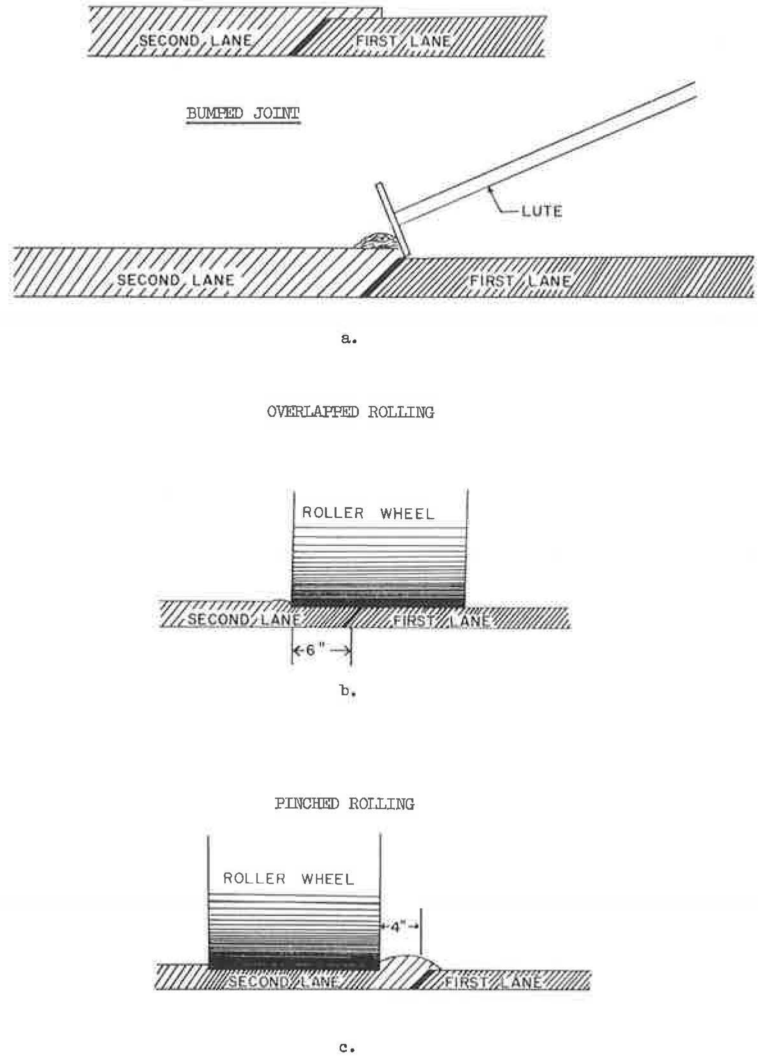


Figure 2.

back with a lute creating a bump (Fig. 2a). In the latter case, the overlap must be adjusted so that the roller can crowd the bump into the joint. Rolling may be started from the first lane overlapping the subsequent lane by about 4 in. (Fig. 2b) or on the subsequent lane pinching the material into the joint (Fig. 2c).

Cold Joints

These occur where the first lane has cooled overnight or longer before the next lane is placed or where the first lane is carried so far ahead that the face has cooled to well below 120 F.

First Lane.—The edge of the first lane, which will be a joint, is always compacted during rolling of the first lane. The edge may be left in this condition before placing the second lane or it may be cut back with a saw or a wheel to produce a fresh face. Saws are used to produce a vertical face. The wheel cutter may be operated to produce a face 30° from the vertical. The edge of the first lane may also be tacked or left untacked. In recent years, infrared heaters have been used to heat the joint before placing the subsequent lane.

Subsequent Lane. —Placement and rolling procedures are the same as those for semi-hot joints.

TEST PROGRAM

North Carolina was selected as the initial location of these studies, but because the I-2 surface mix (Table 1) used is quite plastic at early ages at roadway temperatures, it was believed that many of the problems of longitudinal joint construction would not be present. Therefore, the study was moved to Maryland. Despite constant efforts to improve the procedure, some problems occur in longitudinal joint construction with the PC-1-61 surface mixed used in Maryland which is typical of mixes used in many sections of the United States.

TABLE 1

TYPICAL MIX COMPOSITIONS

Mix	Passing Sieve (%)									Asphalt Content[a] (%)	Marshall Criteria			
	5/8 in.[b]	1/2 in.[b]	3/8 in.[b]	No. 4[b]	No. 10[c]	No. 20[c]	No. 40[c]	No. 80[c]	No. 200		Stability (lb)	Flow (0.01 in.)	Voids (%)	Voids Filled
I-2	-	100	98	80	65	48	32	13	7[d]	6.25	600	9-16	4-10	-
PC-1-61	100	99	88	68	45	34	24	12.5	4.4[e]	5.6	1,200-2,500	10-17	2-5	75-85

[a]Penetration grade 85-100.
[b]Crushed stone.
[c]Screenings and sand.
[d]Natural fines and 3% fly ash.
[e]Limestone filler and natural fines.

Construction Methods

Hot Joints. —Hot joints studied in Maryland were made with pavers operating in echelon. The joint between the two lanes was rolled by overlapping. This method normally produces very satisfactory joints and was included as a basis of comparison.

Semi-hot Joints. —The semi-hot joint is not normally permitted in Maryland and was simulated for this case on a job where pavers were being operated in tandem by placing one lane approximately 700 to 800 ft in advance of the second lane. This resulted in temperatures ranging from 120 to 140 F at the joint side of the initial lane at the time the second lane was placed. At two locations, the initial lane was uncompacted; that is, it was not rolled to the edge before placing the second lane. This condition was studied with the joint face tacked and untacked. In all other instances, the initial lane was compacted or rolled to the edge before placing the second lane. This condition was studied with joints pinched, bumped or overlapped; for each method the joint face was tacked and untacked.

Cold Joints. —Cold joints studied in this investigation were constructed by allowing the first lane to cool overnight before the second lane was constructed. Temperature of the face of the joint for the initial lane was approximately 60 to 75 F in the North

Carolina tests and 35 to 45 F in the Maryland studies. In all instances, the initial lane was rolled to the edge before placing the second lane.

Joint Trimming. —Longitudinal joints are trimmed to remove uncompacted, low density material and present a clean, firm face for contact with the paving material placed for the adjoining lane.

The edge of the lane was wheel cut with a unit attached to a grader. This was found to be difficult because the operator could not see the wheel mounted in front of the blade and the large pneumatic tires caused unsteadiness. Such a device mounted on a steel-wheeled roller may be more successful and provide a quick method of making a smooth, continuous cut. Rollers are also more readily available than graders on the average project.

Joints were cut back with a concrete saw for the purpose of comparison. Except in unusual circumstances, saw cutting is not practicable.

All joints in this study were cut to a vertical face. Trimming at an angle somewhat off the vertical might have some value but was not studied in this investigation.

Joint Heating. —Of cold joints constructed using infrared heating, 12 samples were taken from the Capital Beltway in Maryland and 6 samples were obtained from North Carolina.

During the application of heat to the cold joints, it was noted that a pencil could be pushed easily more than $\frac{1}{2}$ in. into the heated material. The 8-ft long heating device was attached to the side of a paver and maintained $1\frac{1}{2}$ in. above the pavement surface. It successfully softened the pavement to a depth of approximately $\frac{3}{4}$ in. for a width of approximately 4 in. at the edge of the cold lane.

Weather and Construction Conditions

Maryland. —Pavement laid under the adverse conditions of late fall is normally more prone to longitudinal joint failure. In recognition of this, the Maryland tests were made in November when ambient temperatures during paving operations ranged from 40 to 50 F and during the cold-joint studies, in which the first lane was allowed to cool overnight before the laying of the second lane, the temperature dipped slightly below the freezing mark. The single exception was the hot-joint construction done on October 24, with the maximum ambient temperature of 54 F.

Hot and semi-hot joint samples were taken from the $1\frac{1}{2}$-in. overlay of PC-1-61 surface course on the eastbound lane of Route 40 in Maryland, approximately 12 mi east of Baltimore. This is a portland cement concrete 4-lane dual highway, carrying an average ADT of more than 27,000 vehicles with 14 percent heavy trucking.

Samples of cold joints were obtained from the Capitol Beltway which encircles the city of Washington, D. C. The cut-off date for paving had already passed when permission was granted by Maryland State Roads Commission to conduct research at this location. A special paving extension was authorized and the paving contractor cooperated in the various construction techniques involved.

Breakdown rolling by both participating contractors was accomplished by use of a 2-wheel, 10 to 12 ton tandem roller, followed by rubber-tired rolling and finish rolling with a 3-axle tandem. Because the major problem appears to be the compaction of the free edge of the lane, the use of a 3-wheeled roller would probably not have greatly affected the findings. This is confirmed by the results of the North Carolina tests.

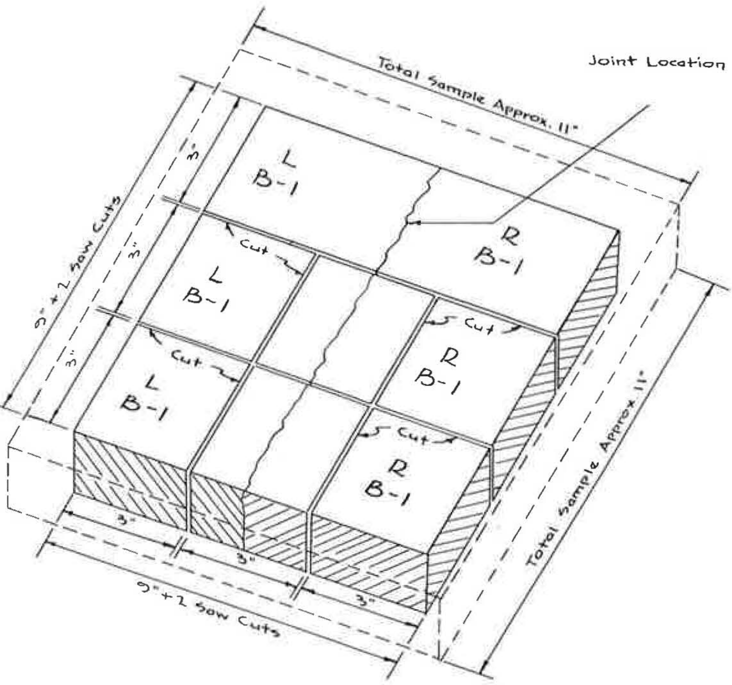
North Carolina. —The cold-joint samples were taken from the westbound lane of US 70 between Raleigh and Garner. This lane was constructed by reinforcing the existing asphaltic pavement with 3 in. of black base surfaced with two 1-in. courses of North Carolina I-2 mix. All samples were taken from the bottom course of I-2.

A 10 to 12 ton, 3-wheeled roller was used for breakdown, followed by a rubber-tired roller with about 60 psi contact pressure, and an 8 to 12 ton tandem for finish rolling.

All samples were taken the week of September 9, 1962. Although pavement temperatures were fairly low in the early morning, the I-2 surface mixture tended to become plastic when exposed to the heat of the Sun.

Sampling Procedure

All samples were taken from pavement laid under normal construction conditions.



SAMPLE CUTTING PROCEDURE

NOT TO SCALE

Figure 3.

Sections approximately 12 in. square were sawed at predetermined locations, each representing a particular method of joint construction. Locations were also established 6 ft on each side of the joint for comparative samples.

To avoid distortion of or damage to samples during removal, a plane of separation from the tacked surface being covered was provided by taping a sandwich of two layers of aluminum foil separated by one layer of kraft paper to the existing pavement. Sample locations were marked before paving and the exact joint location was carefully referenced immediately following the laying of the first lane. After sawing and removal, each sample was carefully marked with control number, joint location and left and right sides.

For transport from field location to the Maryland State Roads Commission Laboratory, the samples were securely taped, wearing surface down, to $^5/_8$-in. plywood pallets, 12 by 36 in., each holding three samples. All samples were stored on these pallets at room temperature (65 to 77 F).

Each large sample was cut into six 3- by 3-in. coupons (Fig. 3), two at joint location and two from each side, and one 3- by 9-in. coupon, with joint construction midway of the long measurement.

Tests Conducted

Density. —The 3-in. square coupons, representing the joint area and lane edges immediately on either side of this area, were used to determine densities (Table 2). The density of all Maryland samples was determined by the Maryland State Roads Commission Bureau of Materials and Research. Samples were weighted, uncoated, in air and water. Because of the character of this surface mix, practically no absorption

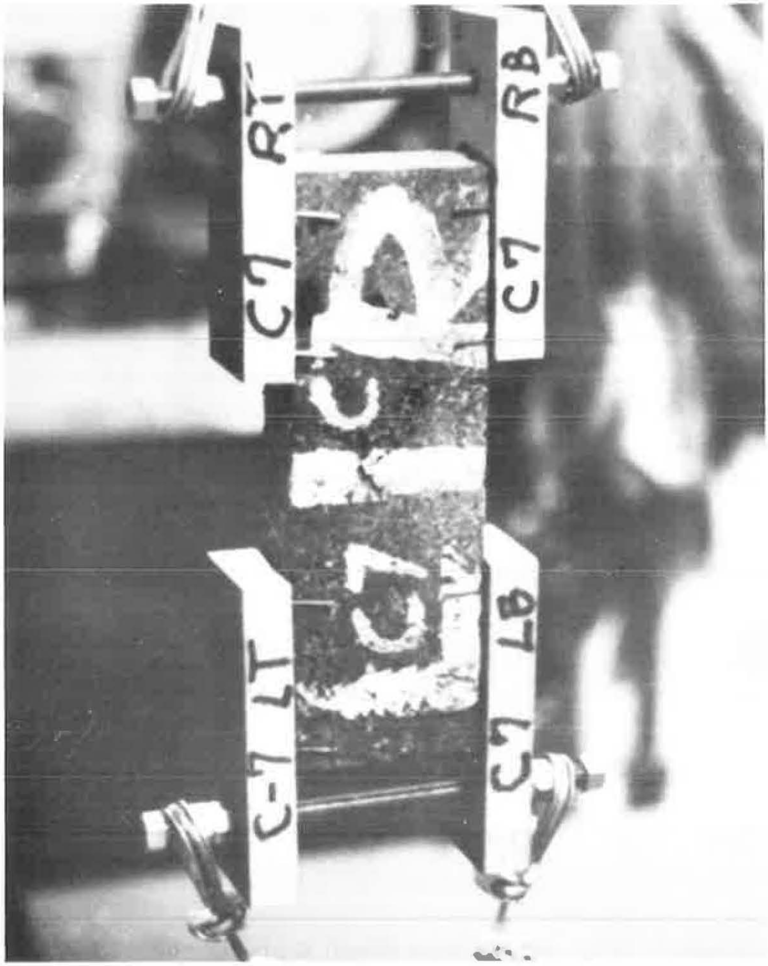


Figure 4. Sample in tensile test.

occurred and coating of the samples with wax was not considered necessary. The results of these tests were the basis for comparing densities within the 9 in. of joint construction area in three sections--left of, right of, and at the joint. In most cases, duplicate samples were available and the results of the two density tests were averaged to represent a given location. Density determinations were also made on lane samples taken 6 ft from the joint for comparison.

Tensile Strength. --Tensile strength tests of bituminous mixes are not a standard procedure but are not entirely without precedent. Massachusetts Institute of Technology used a similar method in testing mixes containing asbestos and some results of tensile tests are available in the literature.

To determine the optimum rate of loading and establish an effective means of applying the load, several 3- by 9-in. coupons of a similar mix were used for developing the test method. Wooden blocks, $\frac{3}{4}$ by 2 by 5 in., were glued to each of the sawed sides of the coupons with an epoxy-type black surfacing compound for pavements, leaving a 3-in. space between the ends of the blocks. These blocks extended approximately 2 in. beyond each end of the coupon and holes were bored to permit the insertion of a $\frac{3}{8}$-in. threaded steel rod. By use of an attached cable yoke, these samples were then tested for tensile strength in a Universal machine (Fig. 4).

The rate of strain was 0.05 in./in./min. All samples were maintained at room

TABLE 2

DENSITIES OF HOT-MIX ASPHALT SAMPLES

Sample No.	Type of Joint	Density (pcf): Adjacent to Joint Unconfined Side	Density (pcf): Joint	Density (pcf): Adjacent to Joint Confined Side	Density (pcf): Lane Samples
	Hot Joints - Maryland				
A-1	Overlapped Rolling	142.5	142.8	144.1	
A-2	Overlapped Rolling	144.5	144.6	143.0	
A-3	Overlapped Rolling		140.6		
A-4	Lane Sample for Comparison				137.9
A-5	Lane Sample - Density only				142.8
A-6	Lane Sample - Density only				147.0
	Semi-hot Joints - Maryland				
B-1	Not tacked, Not compacted	138.1	136.4	147.0	
B-2	Not tacked, Compacted, Pinched	139.0	141.6	147.0	
B-3	Not tacked, Compacted, Bumped	137.3	141.9	148.3	
B-4	Not tacked, Compacted, Overlapped	143.2	140.2	148.4	
B-5	Tacked, Not compacted	136.9	136.7	147.2	
B-6	Tacked, Compacted, Pinched	141.1	144.6	146.4	
B-7	Tacked, Compacted, Bumped	138.5	144.5	147.4	
B-8	Tacked, Compacted, Overlapped	143.8	139.7	148.7	
B-15	Not tacked, Compacted, Overlapped	143.8	141.2	141.2	
B-16	Not tacked, Compacted, Overlapped	145.8	146.2	145.3	
B-17	Lane Sample - Density only				140.4
B-18	Lane Sample - Density only				139.5
B-19	Lane Sample - Density only				144.5
B-21	Lane Sample - Density only				140.2
	Cold Joints - Maryland				
C-1	Not tacked, Compacted, Pinched	136.7	134.9	145.9	
C-2	Not tacked, Compacted, Bumped	137.6	137.9	142.0	
C-3	Not tacked, Compacted, Overlapped	136.5	140.4	147.6	
C-4	Tacked, Compacted, Pinched	140.0	140.3	146.9	
C-5	Tacked, Compacted, Bumped	137.1	140.7	145.3	
C-6	Tacked, Compacted, Overlapped	136.3	138.1	143.2	
C-7	Not tacked, Pinched, I-R Heated	137.5	135.8	143.7	
C-8	Not tacked, Bumped, I-R Heated	133.2	138.3	145.1	
C-9	Not tacked, Overlapped, I-R Heated	133.7	136.7	140.4	
C-10	Tacked, Pinched, I-R Heated	137.0	138.7	144.0	
C-11	Tacked, Bumped, I-R Heated	137.3	132.1	142.7	
C-12	Tacked, Overlapped, I-R Heated	136.8	131.8	140.9	
	Cold Joints - Maryland - continued				
C-13	Tacked, Compacted, Pinched, Sawed	139.4	140.3	145.6	
C-14	Tacked, Compacted, Bumped, Sawed	138.7	142.3	144.8	
C-15	Tacked, Compacted, Overlapped, Sawed	140.2	139.1	145.0	
C-16	Not tacked, Compacted, Pinched, Wheel Cut	138.3	138.1	145.7	
C-17	Not tacked, Compacted, Bumped, Wheel Cut	137.5	141.5	143.9	
C-18	Not tacked, Compacted, Overlapped, Wheel Cut	140.7	140.2	145.3	
C-19	Not tacked, Pinched, Sawed, I-R Heated	140.8	138.7	143.6	
C-20	Not tacked, Bumped, Sawed, I-R Heated	138.2	141.5	147.6	
C-21	Not tacked, Overlapped, Sawed, I-R Heated	140.1	138.7	143.0	
C-22	Not tacked, Pinched, Wheel Cut, I-R Heated	140.3	138.7	142.6	
C-23	Not tacked, Bumped, Wheel Cut, I-R Heated	137.1	142.5	147.0	
C-24	Not tacked, Overlapped, Wheel Cut, I-R Heated	139.5	138.2	144.7	
C-25	Tacked, Compacted, Bumped, Wheel Cut	139.3	135.3	139.3	
C-26	Tacked, Compacted, Overlapped, Wheel Cut	138.0	140.6	147.3	
C-27	Tacked, Compacted, Pinched, Wheel Cut	133.4	137.8	148.0	
C-28	Lane Sample for Comparison				144.6
C-29	Lane Sample for Comparison				145.[illegible]
C-30	Lane Sample for Comparison				144.6
C-31	Lane Sample for Comparison				138.[illegible]
C-32	Lane Sample for Comparison				145.3
C-33	Lane Sample for Comparison				144.[illegible]
	Cold Joints - North Carolina				
D-1	Tacked, Overlapped	125.5	125.9	129.5	
D-2	Not tacked, Overlapped, Bumped	126.1	131.2	131.2	
D-3	Not tacked, Overlapped	123.3	125.3	131.5	
D-4	Not tacked, Bumped	124.1	118.9	131.5	
D-5	Not tacked, Overlapped	125.5	127.9	132.0	
D-6	Not tacked, Overlapped, Bumped	122.7	123.7	129.7	
D-7	Not tacked, Overlapped	125.2	120.2	126.5	
D-8	Tacked, Overlapped, Bumped	121.7	121.6	130.5	
D-9	Tacked, Overlapped	122.7	126.1	129.5	
	Cold Joints - North Carolina - continued				
D-10	Tacked, Overlapped, Bumped	122.1	123.5	129.5	
D-11	Tacked, Overlapped	123.5	124.4	128.2	
D-12	Tacked, Bumped, Pinched	123.8	128.2	130.0	
D-14	Tacked, Bumped, Pinched	123.6	131.7	128.0	
D-15	Not tacked, Overlapped	126.5	125.4	131.0	
D-16	Not tacked, Overlapped, I-R Heated	127.9	126.8	130.6	
D-17	Not tacked, Overlapped	126.5	130.6	132.6	
D-18	Not tacked, Overlapped, I-R Heated	126.9	127.5	132.0	
D-19	Not tacked, Overlapped, I-R Heated	124.6	129.7	131.8	
D-20	Not tacked, Overlapped	126.4	132.1	133.5	
D-1c	Lane Sample for Comparison				124.0
D-7c	Lane Sample for Comparison				129.2
D-8c	Lane Sample for Comparison				124.6
D-14c	Lane Sample for Comparison				125.4

TABLE 3

MAXIMUM TENSILE STRENGTH OF HOT-MIX ASPHALT SAMPLES

Sample No.	Type of Joint	Maximum Tensile Strength (psi): Location of Failure: Adjacent to Joint Unconfined Side	Joint	Adjacent to Joint Confined Side	Indefinite	Lane Sample
	Hot Joints - Maryland					
A-1	Overlapped Rolling	11.0				
A-2	Overlapped Rolling			17.1		
A-4	Lane Sample for Comparison					9.7
	Semi-hot Joints - Maryland					
B-1	Not tacked, Not compacted		10.1			
B-2	Not tacked, Compacted, Pinched	13.5				
B-3	Not tacked, Compacted, Bumped		14.5			
B-4	Not tacked, Compacted, Overlapped		13.7			
B-5	Tacked, Not compacted			13.4		
B-6	Tacked, Compacted, Pinched		10.7			
B-7	Tacked, Compacted, Bumped		11.1			
B-8	Tacked, Compacted, Overlapped		13.3			
B-15	Not tacked, Compacted, Overlapped			12.5		
B-16	Not tacked, Compacted, Overlapped			14.3		
	Cold Joints - Maryland					
C-1	Not tacked, Compacted, Pinched	12.4				
C-2	Not tacked, Compacted, Bumped				17.1	
C-3	Not tacked, Compacted, Overlapped	9.3				
C-4	Tacked, Compacted, Pinched	17.1				
C-5	Tacked, Compacted, Bumped	14.2				
C-6	Tacked, Compacted, Overlapped	15.1				
C-7	Not tacked, Pinched, I-R Heated				15.9	
C-8	Not tacked, Bumped, I-R Heated	14.1				
C-9	Not tacked, Overlapped, I-R Heated	12.0				
C-10	Tacked, Pinched, I-R Heated	16.3				
C-11	Tacked, Bumped, I-R Heated				13.9	
C-12	Tacked, Overlapped, I-R Heated		11.1			
C-13	Tacked, Compacted, Pinched, Sawed		16.3			
C-14	Tacked, Compacted, Bumped, Sawed		15.5			
C-15	Tacked, Compacted, Overlapped, Sawed		20.1			
C-16	Not tacked, Compacted, Pinched, Wheel Cut		18.7			
C-17	Not tacked, Compacted, Bumped, Wheel Cut	19.4				
	Cold Joints - Maryland - continued					
C-18	Not tacked, Compacted, Overlapped, Wheel Cut		18.4			
C-19	Not tacked, Pinched, Sawed, I-R Heated		2.4			
C-20	Not tacked, Bumped, Sawed, I-R Heated	17.1				
C-21	Not tacked, Overlapped, Sawed, I-R Heated		11.1			
C-22	Not tacked, Pinched, Wheel Cut, I-R Heated		4.6			
C-23	Not tacked, Bumped, Wheel Cut, I-R Heated	22.9				
C-24	Not tacked, Overlapped, Wheel Cut, I-R Heated		16.8			
C-25	Tacked, Compacted, Bumped, Wheel Cut		13.1			
C-26	Tacked, Compacted, Overlapped, Wheel Cut		17.9			
C-27	Tacked, Compacted, Pinched, Wheel Cut		15.1			
C-28	Lane Sample for Comparison					38.9
C-29	Lane Sample for Comparison					40.4
C-30	Lane Sample for Comparison					39.2
C-31	Lane Sample for Comparison					23.9
C-32	Lane Sample for Comparison					39.0
C-33	Lane Sample for Comparison					36.9
	Cold Joints - North Carolina					
D-1	Tacked, Overlapped		15.0			
D-2	Not tacked, Overlapped, Bumped	29.0				
D-3	Not tacked, Overlapped	7.0				
D-4	Not tacked, Bumped		4.5			
D-5	Not tacked, Overlapped				6.5	
D-6	Not tacked, Overlapped, Bumped				3.0	
D-7	Not tacked,				6.0	
D-8	Tacked, Overlapped, Bumped	12.0				
D-9	Tacked, Overlapped	19.0				
D-11	Tacked, Overlapped	8.1				
D-12	Tacked, Bumped, Pinched	12.0				
D-13	Tacked, Pinched		7.0			
D-14	Tacked, Bumped, Pinched	8.0				
D-15	Not tacked, Overlapped	27.5				
D-16	Not tacked, Overlapped, I-R Heated	25.0				

temperature (70 to 75 F) during storage, and ambient temperature during testing was in the same range.

The location of the first crack and the load applied at its appearance were determined. In many cases, the samples failed at locations other than at the joint. Loading was continued to maximum, which was the value used in all data comparisons. Sample measurements, to the nearest $^1/_{16}$ in., were used in computing areas used in reducing total load to pounds per square inch (Table 3).

TEST RESULTS

Density Gradient

An unanticipated finding of these studies is the presence of a severe density gradient across the joint in cold-joint construction and in certain types of semi-hot joint construction (Fig. 5). Also, the area of low density is in the edge of the lane placed first, whereas practically all the special joint construction procedures, such as bumping or pinching, are concerned with attempts to get a high density at the joint in the lane placed subsequently. The extent of the low and high density areas was not determined in these studies and the density gradients shown in Figure 5 are based on judgment.

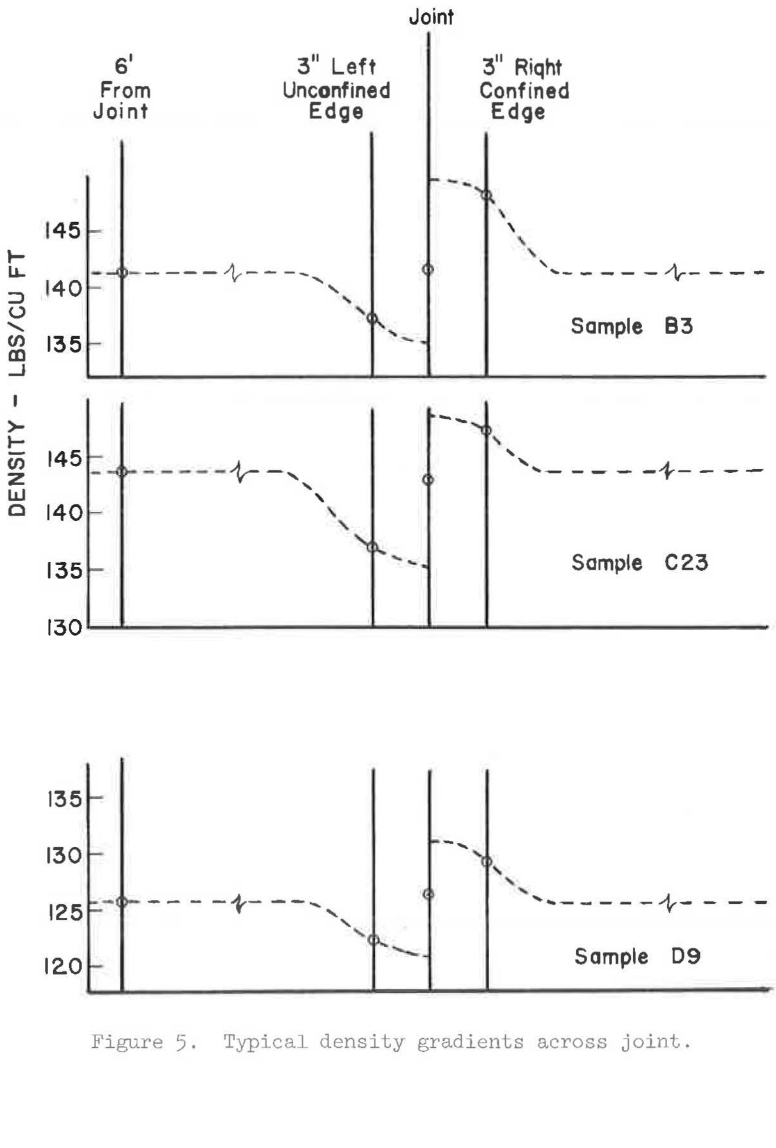


Figure 5. Typical density gradients across joint.

TABLE 4

AVERAGE DENSITIES OF SEMI-HOT JOINTS

Position	Density[a]							
	Overlapped		Pinched		Bumped		Uncompacted	
	Pcf	%	Pcf	%	Pcf	%	Pcf	%
3 in. left	144.2	102.1	139.6	99.1	137.9	97.6	137.5	97.4
Joint	141.8	100.4	143.1	101.4	143.2	101.4	136.5	99.7
3 in. right	145.9	103.3	146.7	103.9	147.9	104.7	147.1	104.2

[a]Percentages based on average lane density of 141.2 pcf.

This unanticipated low density in the unconfined edge may well be the cause of the inability to compact the edge of the first lane properly with present equipment. This low density, as brought out in this analysis, is not solved by tacking or infrared heating. Bumping and pinching the joint on the opposite side does not correct the condition and may serve to accentuate the problem in that it creates a hard spot from the material crowded in at this point. From a theoretical standpoint, a uniform gradient at the same density level as the rest of the pavement is desirable. Areas of low density are subject to more rapid deterioration from oxidation and may even permit entrance of water. Areas of high density may cause a stress concentration during thermal expansion and contraction and induce cracking. Because the low density appears to be the more severe problem, the analyses of the density data in this report are concerned primarily with the values measured 3 in. left of the joint in the unconfined edge.

Hot Joints

Observations and density tests of the hot joints indicated that there was virtually no longitudinal joint as such, and the lanes were in essence a single entity. Average densities at 3 in. left, 3 in. right, and at the joint were 143.5, 143.6, and 142.7 pcf or, in terms of percent considering 142.6 pcf the average of the lane samples, 100.5, 100.6, and 99.9, respectively.

Tensile test results of the hot joint were higher than those from the lane sample (Table 3); however, the density of the lane sample was relatively low so this comparison should not be given much weight.

Semi-hot Joints

Semi-hot joints, as might be expected, represent a compromise between hot and cold joints. Table 4 summarizes the densities measured at the semi-hot joints. The use of a tack coat was not considered in the measurements because it should have very little effect on density. It will be noted, that the overlapped joint showed significantly higher densities 3 in. left of the joint than the other methods. Densities 3 in. right of the joint were much greater than the average of the lane samples in all cases. Comparison of the tests where the sample failed at the joint (Table 3) indicates that the tensile strength of untacked joints was of the order of or higher than samples with tack. Also, of the six untacked samples tested, three failed at locations away from the joint, whereas of the four tacked samples tested, three failed at the joint. It is apparent that the tack provided no improvement in the tensile strength; however, it may be effective in improving resistance to infiltrating water. This feature was not studied.

Maryland Densities. —An inspection of the density values 3 in. left of the joint (Table 2) shows those of Samples C8, C9, and C27 were much less than the other values for no apparent reason, and these have not been used. Also, the primary variable affecting the density 3 in. left of the joint was whether or not the joint was cut back. Therefore, in these analyses, all values for cut joints are compared to averages for uncut joints to show the effect of a specific variable.

Infrared heating of the joint had a small effect on the density 3 in. left of the joint when the joint was cut back (139.1 pcf with heating vs 138.1 pcf without) but showed no effect where the joint was not cut back (137.1 pcf with heating vs 137.5 pcf without).

At 3 in. left of the joint sawing produced very slightly higher densities (0.5 to 0.8 pcf) than wheel cutting. There was no consistant variation in the density for overlapped, bumped, or pinched joints.

Average densities of all cut and uncut joints are compared in Table 5. Cutting back the joint increased density at 3 in. left of joint about 2 pcf. The lane density in these tests is about 4 to 5 percent less than that found in the tests on hot and semi-hot joints in Maryland and on cold joints in North Carolina. No explanation can be given for this, because the spread is about 5 percent between left and right in all cases.

TABLE 5

AVERAGE DENSITIES OF CUT AND UNCUT JOINTS

Position	Density[a] Cut Back Pcf	Cut Back %	Not Cut Back Pcf	Not Cut Back %
3 in. left	139.2	96.8	137.3	95.5
Joint	139.6	97.1	137.1	95.3
3 in. right	144.9	100.8	144.0	100.1

[a]Percentages based on average lane density of 143.8 pcf.

Maryland Tension Tests.—Figure 6 is a plot of tensile strength vs density. Samples C-19 and C-22 failed at a crack present in the sample before the test was started and these values are not used. Also, three samples were not used because the density values seemed too low. Results of tests that failed at the joint are shown in large symbols. The points that plot well above the curve, indicating lower tensile strengths than nonjointed material of the same density, are all samples from joints cut back with either the wheel or the saw; however, some samples from joints that were cut back show strengths equal to nonjointed material. Cutting back the joint may, but not necessarily will, produce a low tensile strength. Figure 6 indicates that the tack coat (solid symbols) apparently increased the tensile strength for sawed joints but not for wheel-cut joints. Of samples that failed at the joint, nine were tacked and five were not; of those that failed away from the joint, four were tacked and seven were not. This indicates that the tack was not effective in increasing tensile strength.

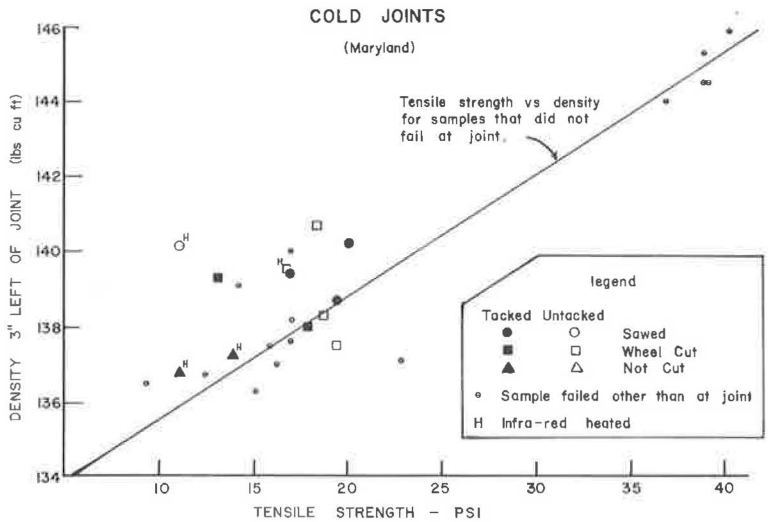


Figure 6.

TABLE 6

AVERAGE DENSITIES OF NORTH CAROLINA SAMPLES

Treatment	Density[a]					
	3 In. Left		Joint		3 In. Right	
	Pcf	%	Pcf	%	Pcf	%
Overlapped and I-R heated	126.5	100.6	128.0	101.8	131.5	104.6
Overlapped or bumped	125.0	99.4	126.2	100.2	130.6	103.8
Bumped plus overlapped or pinched	122.8	97.6	125.7	99.9	129.5	103.0

[a]Percentages based on average lane density of 125.8 pcf.

The data plotted on Figure 6 do not indicate that infrared heating was effective in increasing the tensile strength of cut joints.

North Carolina Densities. —Average densities from tacked and untacked cases are given in Table 6. Infrared heating increased the density 3 in. left of the joint slightly. Bumping and overlapping gave essentially equal results, but combinations of bumping with either pinching or overlapping gave the lowest densities.

North Carolina Tension Tests. —Figure 7 is a plot of tensile strength vs density. The two cases plotted where the tests failed at the joint show tensile strengths a little less than those of samples not failing at the joint. One of these joints was tacked; the other was not. Other evidence that the tack coat was not particularly effective in increasing tensile strength is that two of the three samples that failed at the joint were tacked, whereas seven of the thirteen samples that failed away from the joint were not tacked. Only one heated joint was tested (D-16). It did not fail at the joint, but the tensile strength was no higher than a companion (D-15) unheated joint.

DISCUSSION

Because bituminous surface mixture has been observed to move laterally under breakdown rolling, the resulting low density at the edge of the lane appears logical. How far this low-density area extends from the joint and what can be done to correct the weakness remain questionable.

It is not essential to know the extent of the low-density area if truly confined rolling can be accomplished. For example, if forms are used to confine the initial lane, the need for further defining the problem area is unnecessary. However, an approach

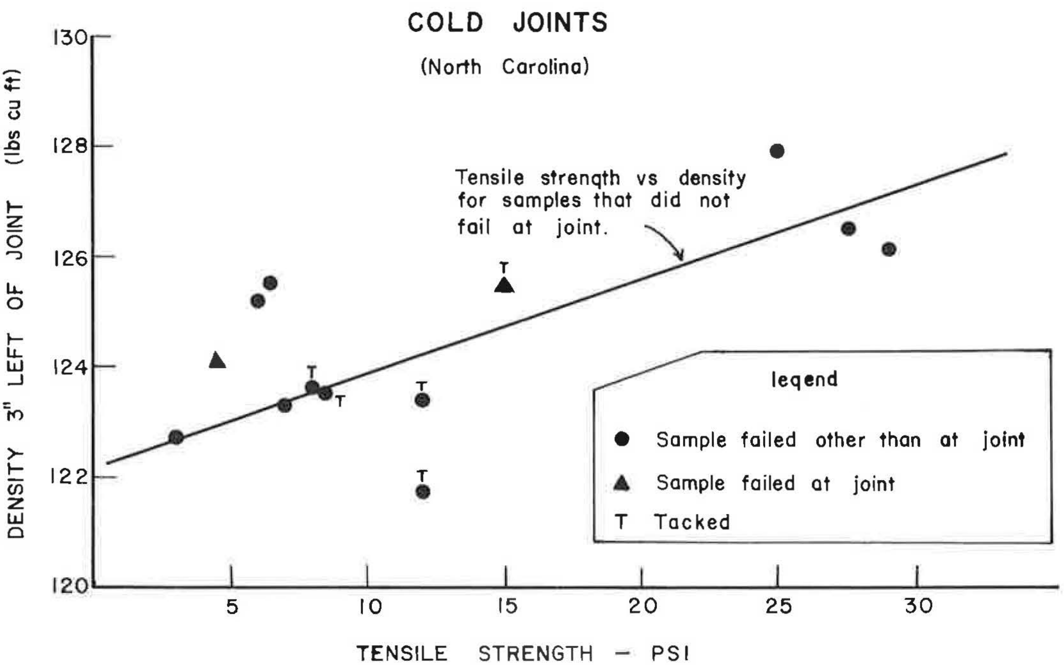


Figure 7.

successfully accomplishing the purpose might be either impractical or economically unsound. Defining the extent of the low density area would make possible a number of alternate approaches to a more reasonable solution.

One further point should be made in this regard. If the exact location of the joint had not been carefully defined before paving the second lane in these studies, many joints would have been assumed to be located 2 to 3 in. to the cold side of the true location. This apparent shift is generally caused by the normal overlap of second-lane material onto the first. Material is pushed back off the cold pavement, but the evidence of this overlap remains. Several times, workmen insisted that the reference lines were incorrect when samples were being cut. Subsequent visits to the test areas showed that the incorrect evidence of joint location remained. Therefore, much of the so-called joint failure apparently centered over the joint may in fact be entirely in the uncompacted area in the unconfined lane.

This, then, is the problem. Rolling a bituminous surface mix in a plastic state without edge confinement cannot produce the density designed or required. When the resulting pavement cools before the adjoining lane is placed, an inherent weakness is left in the area extending from the joint for an unknown distance into the first lane paved. Eventually, some form of confinement, edge compaction, infrared heating or a combination of these may provide a workable solution.

FINDINGS

This investigation found that a low density zone at the edge of the initial lane and a high density zone at the edge of the subsequently laid lane exist that are not present in hot joints made with pavers operating in echelon. In addition, the results show that:

1. Overlapped rolling produced the highest densities in the initial lane in semi-hot joint construction;
2. There was no clear-cut superiority to any of the procedures used in cold-joint construction from the standpoint of density in the initial lane except that combinations of bumping and pinching and bumping and overlapping produced the lowest density;
3. Cutting back the joint improved density in the initial lane slightly but caused low tensile strengths;
4. Infrared heating improved density slightly in the initial lane but did not improve tensile strength;
5. Tack on the edge of the joint did not improve tensile strength.

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