HIGHWAY RESEARCH BOARD
Bulletin 154

Performance of Bituminous Surfacing

National Academy of Sciences—
National Research Council
publication 489

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Thirty-Fifth Annual Meeting
January 17–20, 1956

1957
Washington, D. C.

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Contents

Field and Laboratory Investigations on Hot Mix Asphalt Concrete

JOHN W. SHUPE, Associate Professor of Applied Mechanics, and
DELOS C. TAYLOR, Associate Professor of Applied Mechanics
Kansas State College, Manhattan, Kansas

● THE Applied Mechanics Department of Kansas State College and the State Highway Commission of Kansas are engaged in a cooperative study to investigate the behavior of hot mix asphalt concrete under the action of weathering and traffic. The primary purpose of this research program is to develop a better correlation between the laboratory tests and the actual performance of the asphalt concrete as a pavement surface. A secondary objective of this study is to evaluate the effect of commercial anti-stripping agents upon the properties of hot mix asphalt concrete.

Two series of test sections were placed in the Fall of 1952. This report contains a description of the test sections, an outline of the sampling and testing procedures, and the test results obtained on the original material as well as on samples removed from the highway surface after one, two, and three years of service. Comparison of the test sections will continue for the useful life of the pavements.

Description of Test Sections

The first test series was placed September 23, 1952, on a 4-lane section of US Highway 81 located two miles south of Newton, Kansas. The asphalt concrete was made with chat aggregate containing approximately seven percent AC-5. The construction project consisted of laying a 22-ft pavement over the westerly two lanes of the old portland cement concrete highway connecting Newton and Wichita. The surface was placed in two 11-ft strips with each strip containing two lifts of approximately 1.25 in. each, giving a total slab thickness of from 2.5 to 3 in. The 1953 Kansas Highway Sufficiency Rating Survey listed the average daily traffic for the two southbound lanes at 8,600 vehicles per day.

Two commercial anti-stripping agents were used, and will be referred to as Additive "A" and Additive "B." Three test sections were placed containing, respectively, no additive, 1.5 percent Additive "A," and 1.5 percent Additive "B." The percent of additive was based on the weight of the asphalt.

Subgrade conditions were nearly identical throughout the length of the test sections, so it was felt that a better duplication of conditions could be achieved if the three test sections were located in the same traffic lane rather than adjacent lanes. The outside lane was selected in order to subject the test surface to the most severe traffic conditions.

Each test section was slightly over 700 ft long, and required a total of 16 truck loads of treated material to supply both the leveling and surface courses.

The second test series was placed October 22, 1952, on a 2-lane section of US Highway 40, immediately northeast of Fort Riley, and within the city limits of Ogden, Kansas. Two months after construction this section was designated as an alternate route to US 40. The average daily traffic count on this highway in 1953 was 4,840 vehicles per day.

This project also consisted of placing a 22-ft pavement over two lanes of old portland cement concrete, and the construction procedure was identical with that described for the first test series. However, the bituminous mixture deviated from that of the Newton Project in two respects; namely, limestone aggregate was used in place of chat, and the amount of additive in the treated section was reduced from 1.5 percent to 1.0 percent of the total weight of the asphalt.

As in the first test series, the original plan was to have 700-ft test sections, and the leveling course was laid satisfactorily in this manner. Unfortunately, due to a misunderstanding with the contractor when the surface course was placed, the treated sections in the surface course failed to coincide with those of the leveling course by a distance of nearly 500 ft. As a result, instead of having three 700-ft test sections, there were three

TABLE 1

MATERIALS IDENTIFICATION FOR THE TWO TEST SECTIONS

Ogden Test Section—(Limestone Aggregate)

	Theoretical Mix		Extracted Samples—3 years		
			W/O Add.	Add. A	Add. B
Course	Leveling	Surface	Combined	Combined	Combined
% Asphalt (AC-5)	7.2	7.3	6.6	7.0	7.5
% Retained on sieve					
3/4	2	2	4	2	1
3/8	21	22	15	12	11
4	49	44	43	41	41
8	64	58	60	57	60
16	72	65	69	67	69
30	80	74	77	75	76
50	87	86	81	81	83
80	89	88	84	83	84
100	90	90	85	84	85
200	93	94	88	88	91

Newton Test Section—(Chat Aggregate)

	Theoretical Mix		Extracted Samples—3 years		
			W/O Add.	Add. A	Add. B
Course	Leveling	Surface	Combined	Combined	Combined
% Asphalt (AC-5)	6.5	7.5	7.1	6.9	7.0
% Retained on sieve					
3/4	0	0	0	0	0
3/8	0	0	0	0	0
4	24	23	23	24	25
8	48	47	49	49	49
16	66	65	67	66	65
30	78	76	77	76	75
50	84	82	83	82	81
80	89	86	87	85	85
100	91	87	89	88	87
200	94	91	92	91	90

test sections about 200 ft long, and the remaining 1,500 ft was a mixture of treated and untreated material in the leveling and surface courses.

These 200-ft test sections are of sufficient length for obtaining samples from the pavement, and for visual inspection. They are a little short, however, for determining an accurate road-roughness measurement, which is one of the methods being used to check the progressive failure of the test section.

Table 1 gives a description of the materials used in the asphalt concrete for both test sections. The data for the extracted samples were taken on specimens which had been in service for three years as a highway surface. The extracted specimens were for the full pavement depth; so should give values for both percent asphalt and gradation which are approximately midway between the values determined for the leveling and for the surface courses.

The extracted samples from the Newton project showed excellent consistency not only between each other, but also with the theoretical mix. This was true for both the aggregate gradation and the percent asphalt. The maximum deviation in asphalt content from the theoretical mix was only 0.1 of one percent.

This situation did not exist on the Ogden project, however. There was a much higher percent of fine material than was listed for the theoretical mix, possibly due to the degradation of the relatively soft limestone aggregate over three years of service. There was also a large difference in the asphalt content of the various sections, with the deviation between samples reaching 0.9 of one percent and the maximum deviation from the theoretical mix equaling 0.65 of one percent.

Figure 1 is a photograph of samples sawed from each of the test sections. The two blocks on the right illustrate the harsh, angular chat used on the Newton project, while the other two specimens show the somewhat coarser and more rounded limestone

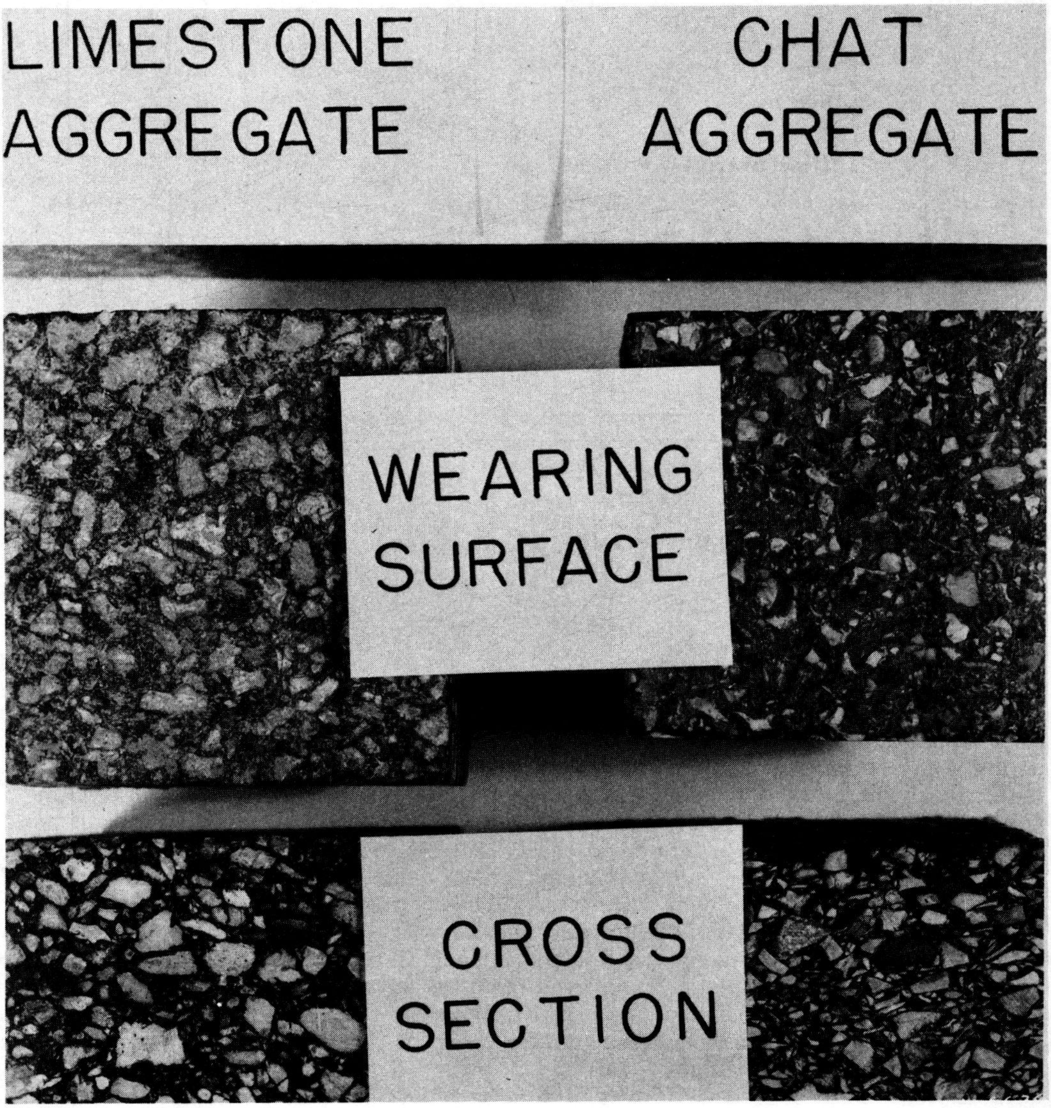

Figure 1.

aggregate used on the Ogden test sections.

Test Samples Removed from the Pavement Surface

To facilitate ease in obtaining samples from the highway surface, double strips of aluminum foil were placed directly beneath the leveling course. These double strips were securely stapled around the edges and were placed a short distance in front of the paver. They were held in place by the tack coat; and in spots where this coat had lost its tackiness, additional asphalt was used to firmly anchor down these strips so they could resist the shoving effect of the paver.

The dimensions of the foil strips are shown in Figure 2(A). Each strip was 5.5 ft long and 22 in. wide and was placed transversely to the road with a 6-in. overhang at the edge to aid in location. The strips extended into the roadway about 5 ft, a distance which included the outside wheel lane.

A standard concrete saw was used to remove the sections from the pavement. The asphalt concrete was sawed at temperatures up to 85 deg F, and no difficulty was experienced in sawing either the asphalt concrete made with limestone or that made with chat.

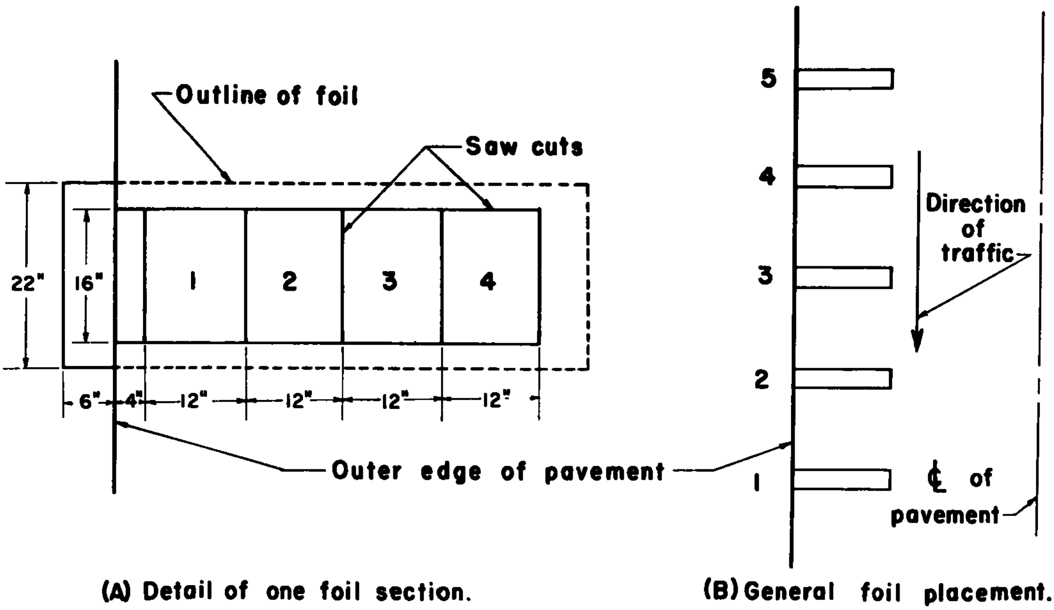

Figure 2. Location and dimensions of foil strips.

The saw cuts were made on the pavement surface as shown in Figure 2(A), with the depth of sawing being sufficient to cut through both foil strips. One layer of foil remained on the tack coat, while the other layer stuck to the under side of the leveling course. The two foil strips acted as a cleavage plane, so that a block of asphalt concrete 12 x 16 in. and 3 in. thick could easily be removed.

The outer three or four inches of the pavement were discarded, and four 12- x 16-in. blocks were obtained from each foil strip. These blocks were further reduced in the laboratory to form the various test specimens.

Five foil strips, 5 ft apart, were placed in each test section as illustrated in Figure 2(B). The first set of samples was sawed from the pavement within three days after the surface course was laid, so that the original series of tests would give an indication of the properties of the asphalt concrete before it had an opportunity to be altered by weathering or the compacting effect of traffic.

This first set of samples was taken from the foil strip farthest along in the direction of traffic (Strip No. 1) so that the bounce effect, if any, due to the patch would not cause

Figure 3. Sawing specimens from the highway.

unequal compaction on the remaining foil strips. Strip No. 2 was removed at the end of one year's service; Strip No. 3 supplied the two-year samples; and Strip No. 4, the three-year specimens.

Figure 3 illustrates the concrete saw in operation. The dark patch in the foreground is where the test specimen was removed the previous year. Figure 4 is a view of the surface after all saw cuts had been made, but before any of the blocks had been disturbed. The circular disc at the outer edge of the pavement was placed there when the surface course was laid, to aid in location of the foil strips at a future date. Figure 5 shows the same section with the outer block removed.

In attempting to obtain samples from sections in which no foil was located, a great deal of difficulty was experienced due to the bond existing between the asphalt concrete and the portland cement concrete base. This is illustrated in Figure 6 which shows an asphalt concrete block sawed from the pavement and inverted. Part of the portland cement concrete base was torn loose and remains attached to the under side of the leveling course. In these sections a specimen only four inches wide could be removed, rather than the 12-in. block which was taken in the foil sections.

In reducing the 12- x 16-in. blocks to laboratory test specimens, the outer three blocks were sawed into test beams approximately 4 inches wide, while the inner block was used to obtain Marshall specimens. Figure 7 illustrates the three types of test specimens obtained from the highway surface.

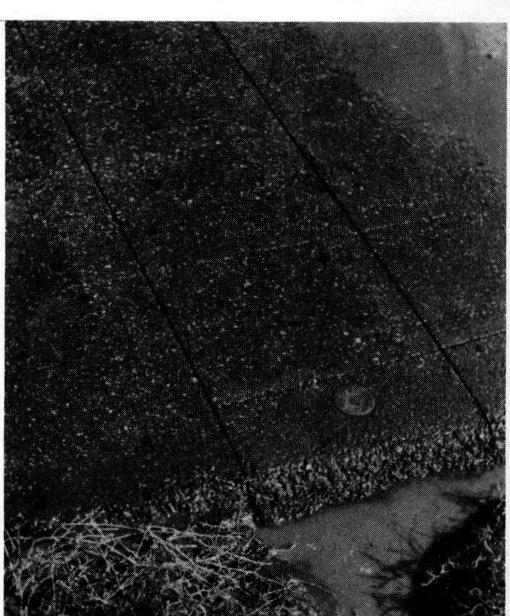

Figure 4. General view of saw cuts.

An Everready "Briksaw" equipped with a 1.5 hp motor was used to saw the blocks into beams. It was also used to saw the beams into unconfined compression samples. Of the variety of blades which were used, a water-cooled abrasive blade containing nylon cord reinforcing gave the best results. At 3,300 rpm this blade was quite satisfactory for sawing both the asphalt concrete made with limestone and that made with chat.

Unfortunately, the procedure used in obtaining Marshall specimens from the pavement did not prove satisfactory for both aggregate types. A drill bit utilizing carboloy chips for teeth, similar to the type described in the Corps of Engineers Technical Memorandum No. 3-254, gave acceptable specimens with the asphalt concrete containing limestone, but not with chat. The samples of chat asphalt were very badly "chewed up," and in some cases there was a complete separation between the leveling and wearing courses. As a result, the data on Marshall specimens taken from the pavement surface are somewhat incomplete.

Figure 5. Outer block removed from pavement.

Specimens Molded in the Laboratory

In an effort to duplicate the material of the specimens molded in the laboratory with that existing on the highway surface, all of the bituminous material used in the laboratory specimens was taken directly from the asphalt paver at the point where it passed over each corresponding foil section. This material was taken from the leveling course. Therefore, in comparing the results on a specimen molded in the laboratory with one sawed from the pavement, it is known that each laboratory sample and the lower half of the corresponding field specimen originally came from the same truck load of material. The samples taken from the paver were placed in gallon buckets and sealed immediately.

There were three types of laboratory specimens molded for each variety of material: three Marshall specimens, six Immersion-Compression specimens, and four asphalt beams. In molding all samples the material was first reheated to 300 deg F.

The Marshall specimens were formed by placing sufficient material in a 4-in. diameter mold to give a finished specimen approximately 2.5 in. thick. This material was then compacted by 50 blows on each side, from a 10-lb hammer dropping through a distance of 18 inches. After aging in air for 18 hours, the specimen was brought up to a temperature of 140 deg F by submersion in a waterbath for 30 minutes, and tested in the standard Marshall equipment.

The Immersion-Compression specimens were formed by placing enough material in a 4-in. diameter mold to give a finished specimen 4 in. deep. This material was then subjected to a static pressure of 3,000 psi, which was held for two minutes. The specimen was removed from the mold, cured for 24 hours, and tested in unconfined compression either before or after immersion.

The asphalt beams were made in a roller-type compactor, which was developed to simulate the compaction that an asphalt pavement receives during construction. Figure 8 is a schematic diagram of the roller compactor.

The essential elements of the roller-compactor are a mold, a movable carriage, a rocker, and a fixed roller. The inside dimensions of the mold are 4 x 16 in., and 4.5 in. in depth, so that enough material may be placed in the mold to give a compacted beam approximately 3 in. deep. The mold rests on the movable carriage, which rolls on steel guides placed on the base of an ordinary hydraulic testing machine.

A static pressure of 100 psi was first applied to the material and held for one minute. Next, the rocker, which is a 3/4-in. steel plate 3 7/8-in. wide, and shaped in the arc of a circle 48-in. in diameter, was placed directly on the material. The roller, a 4-in. diameter roller-bearing, is connected through a clevis to the head of the testing ma-

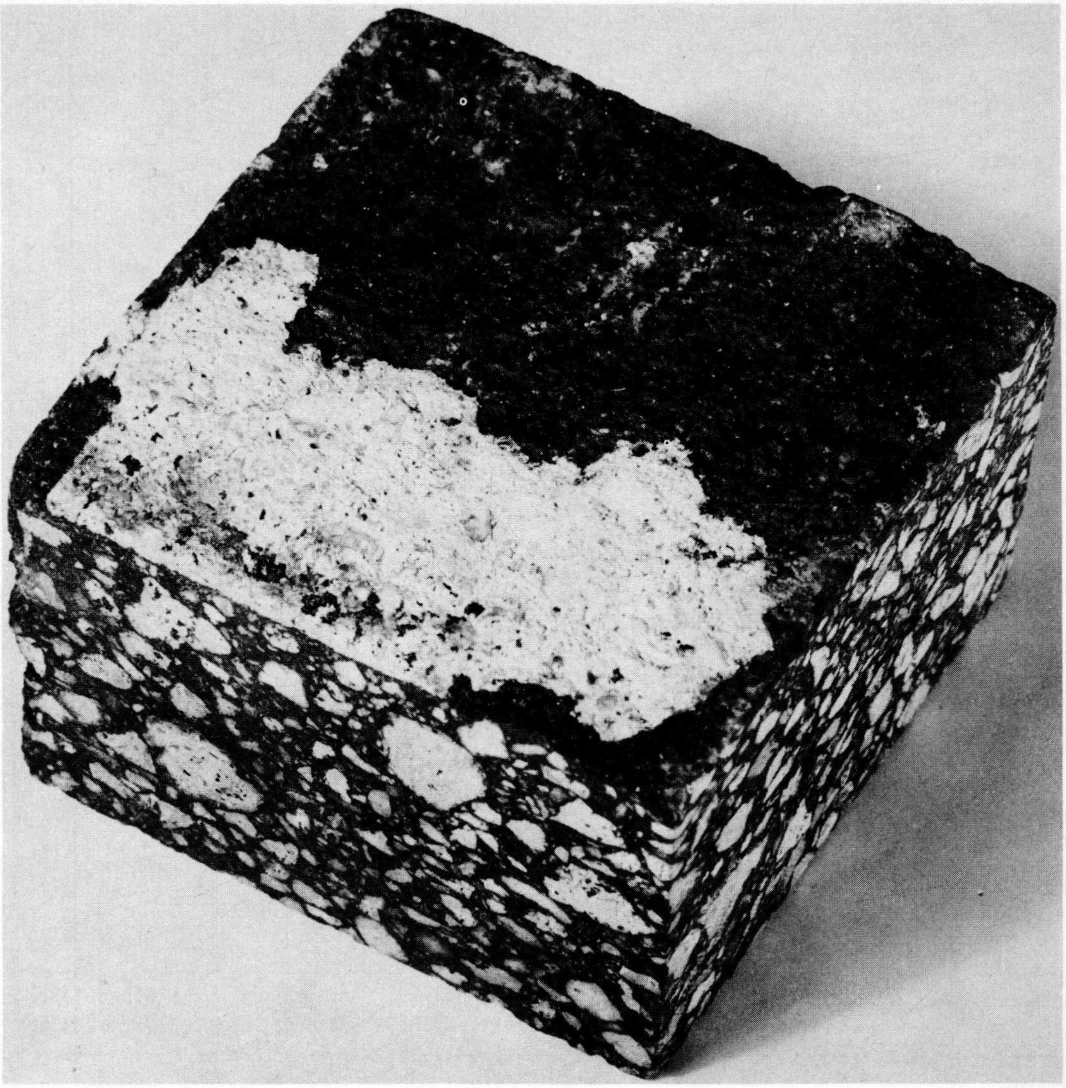

Figure 6. Illustration of bond between base and pavement.

chine. As the head is brought down, the roller exerts a pressure on the rocker which, in turn, exerts a compacting effort on the bituminous mix.

The carriage was pushed manually a total of ten times back and forth under the roller while the load was maintained at 1,200 lb. This load was transmitted to the roller through a large coil spring, which tended to even out the load for all positions of the rocker. This rocking action compacted the material, simulating the behavior of a steel roller on a pavement surface. The finished specimen was a 4- by 16-in. asphalt beam 3 in. deep, that had received the equivalent of 10 passes from a 4-ft diameter, 300 lb per lineal inch steel roller. This was consistent with construction practices in Kansas.

Discussion of Tests and Test Results

A. Specific Gravity: The specific gravity for all specimens was determined in the following manner:

$$S.G. = \frac{A}{B - C}$$

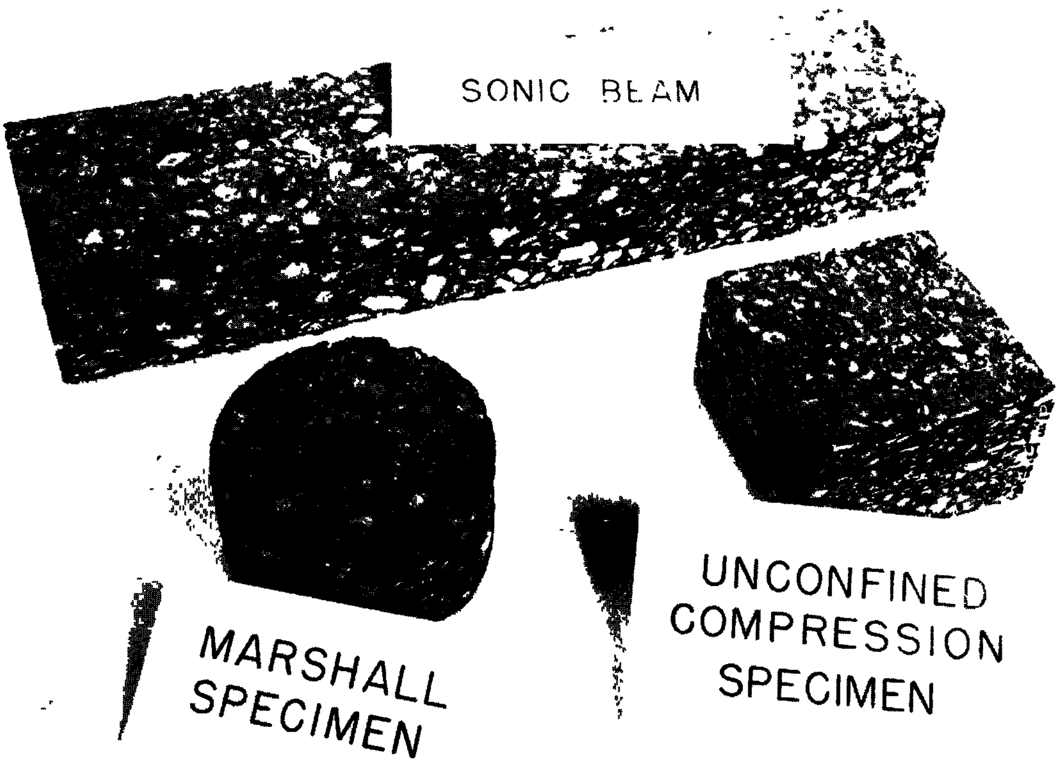

Figure 7.

TABLE 2

SPECIFIC GRAVITY OF TEST SPECIMENS
Specimens Compacted in Laboratory

Aggregate Type	Additive	Marshall (Hammer)	Immersion Compression	Roller Compactor	Maximum Theoretical
Limestone	None	2.38	2.36	2.33	2.39
"	1% A	2.38	2.37	2.33	2.39
"	1% B	2.38	2.36	2.31	2.39
Chat	None	2.13	2.06	2.05	2.37
"	1.5% A	2.11	2.07	2.06	2.37
"	1.5% B	2.13	2.07	2.05	2.37

Specimens Sawed from the Pavement

Aggregate Type	Additive	Original	1 year	2 years	3 years	Maximum Theoretical
Limestone	None	2.27	2.34	2.35	2.36	2.38
"	1% A	2.28	2.35	2.35	2.36	2.38
"	1% B	2.29	2.34	2 35	2.36	2.38
Chat	None	2.02	2.12	2.14	2 13	2.35
"	1.5% A	2.02	2.16	2.18	2.19	2.35
"	1.5% B	2.03	2.15	2 17	2.18	2.35

where: A = dry weight in air
B = weight in air after being submerged for one minute, and surface dried with a damp cloth
C = submerged weight

Table 2 summarizes the results of the specific gravity determinations. Each figure represents the average for 3 Marshall specimens, 6 Immersion-Compressions specimens, 4 beams molded in the roller-compactor, or from 7 to 12 beams sawed from the pavement.

Referring to Table 2, the asphalt concrete containing limestone showed very high densities, with the specimens sawed from the pavement after three years of service giving values over 147 pcf. The Marshall specimens compacted in the laboratory had densities of nearly 149 pcf, which was approaching the maximum theoretical density of the design mix as determined by the Kansas Highway Commission.

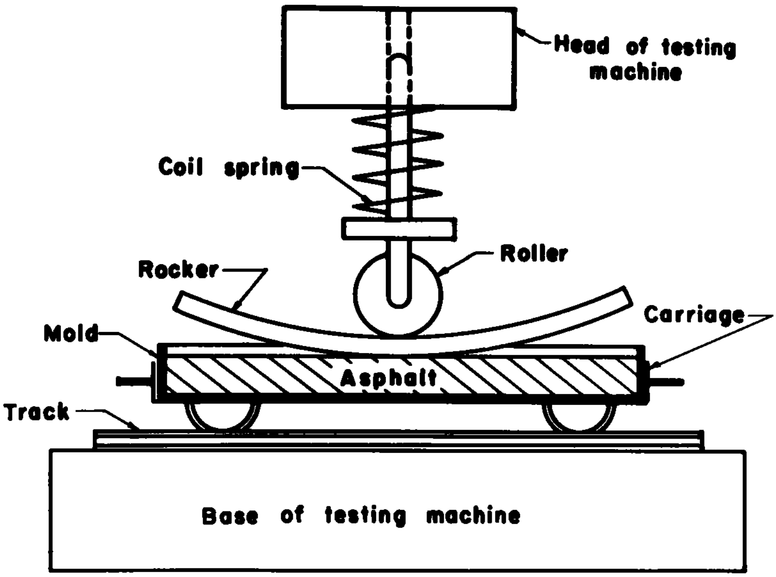

Figure 8. Schematic diagram of the roller-compactor.

The densities of the Immersion-Compression specimens containing limestone aggregate were nearly as high as those of the Marshall specimens and somewhat higher than for the beams made with the roller-compactor. All three laboratory specimens had higher densities than the original samples removed from the pavement.

There was an appreciable increase in the densities of the three test sections of limestone asphalt concrete under the action of traffic. Most of this increase took place in the first year; and, after three years of service, the average increase of the three test sections was 4.2 percent over the original densities. There was no noticeable effect due to the presence of the additives.

The asphalt concrete containing chat had density values which were much lower, with the maximum density noted for any of the test sections reaching a value of 137 pcf. The chat asphalt concrete did exhibit a greater increase in density under the action of traffic, however, with the average increase in density of the three test sections equaling 7.2 percent after three years as a highway surface.

The presence of the additives in the asphalt concrete containing chat seemed to have some effect on the variation in density with time. After three years as a pavement, the section containing no additive increased in density by 5.4 percent; the test section with additive "A" increased by 8.4 percent; and the section with additive "B" by 7.4 percent.

Figure 9 shows the variation in density of two typical surfaces with respect to transverse location in the highway and the length of service as a pavement surface. The asphalt concrete made with limestone contained additive "A" and the chat asphalt concrete contained additive "B." Both surfaces, however, gave results and trends which were representative for their respective aggregate type.

The asphalt concrete made with limestone showed very little change in density with regard to lateral location, particularly after the first year of service. The chat asphalt concrete, on the other hand, showed an increase in density away from the outer edge of the pavement with the maximum density occurring over 2 feet in from the pavement edge. This would reflect, in part, the nature of the traffic on the two test sections. On the Ogden test section the slow speed, 2-lane traffic would utilize nearly the entire pavement width; while on the 4-lane, high speed, rural highway of the Newton test section, there would be no tendency to crowd the outer edge.

B. Marshall Test. In the Marshall test the maximum stability was determined along with the flow or deformation corresponding to the maximum load. Table 3 summarizes the results of the Marshall tests, and indicates the values for the specimens molded in the laboratory and for two sets of field specimens of the asphalt concrete made with limestone. The reason for the scarcity of field data has been discussed previously—no satisfactory method of obtaining cores from the chat asphalt concrete was developed.

The results of Table 3 indicate that for the original material the asphalt concrete containing limestone had much greater stability than that made with chat The flow was also greater for the limestone asphalt, and there seemed to be no apparent additive effect.

The stability values on the samples taken from the highway after two and three years of service were appreciably less, and the flow values much greater than for the original laboratory specimens compacted with the hammer.

C. Immersion-Compression Test. The Immersion-Compression Test was developed by the Bureau of Public Roads to give an indication of the water-resistant characteristics of a bituminous mix. The procedure consists of testing three unconfined compression specimens before and three similar specimens after immersion for four days in a 140 deg F waterbath. (The Bureau recommends 120 deg F, but it was felt that the increased temperature would not be too severe for these high-type mixes, and would emphasize any effect which might arise due to water action.) The results of these tests are shown in Table 4.

The asphalt concrete containing chat showed somewhat greater resistance to water

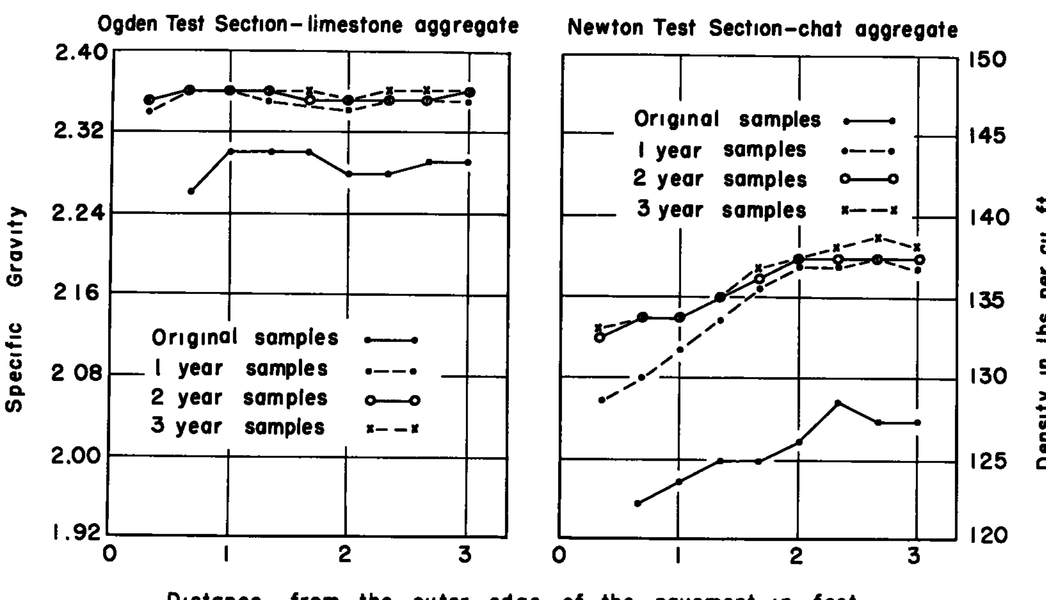

Figure 9. Density variation in test sections.

TABLE 3

MARSHALL TEST RESULTS

Specimens Compacted in Laboratory

Aggregate Type	Additive	Flow in 1/32 inch	Corrected Marshall Stability in lb
Limestone	None	18	2,300
"	1% A	18	2,450
"	1% B	17	2,380
		Av. 18	Av. 2,380
Chat	None	8	1,740
"	1.5% A	11	1,480
"	1.5% B	9	1,550
		Av. 9	Av. 1,590

Specimens Sawed from the Pavement

Aggregate Type	Additive	2 year specimens		3 year specimens	
		Flow	Stability	Flow	Stability
Limestone	None	29	1,400	26	1,210
"	1% A	36	1,370	24	1,100
"	1% B	32	1,410	26	1,120
		Av. 32	1,390	25	1,140

TABLE 4

IMMERSION-COMPRESSION TESTS

Aggregate	Additive	Stress in psi		% Retained Strength
		Before Immersion	After Immersion	
Limestone	None	757	514	68
"	1% A	614	380	62
"	1% B	644	455	71
		Av. 672	Av. 450	Av. 67
Chat	None	265	239	90
"	1.5% A	295	247	84
"	1.5% B	302	266	88
		Av. 287	Av. 251	Av. 87

action, in that the percent retained strengths were higher than for the limestone concrete. However, although the limestone gave the lower retained strength of the two mixes, the actual stress values, both before and after immersion, were appreciably higher than for the asphalt concrete made with chat. There was no noticeable effect due to the presence of the additives.

D. Modified Immersion-Compression Test. This test was patterned after the standard immersion-compression test, with the only difference being in the dimensions and preparation of the test samples. These test specimens were prepared by sawing the 4-by 16-in. beams into four equal lengths and lettering them in order a, b, c, and d. Sections a and c were tested in unconfined compression at 77 deg F; sections b and d were similarly tested after four days immersion in a 140 deg F waterbath. Table 5 lists the results of these tests, and represents the averages for all specimens tested without regard to lateral location in the pavement.

TABLE 5

MODIFIED IMMERSION-COMPRESSION TEST
Samples Sawed from the Pavement

Aggregate Type	Additive	Age	Unconfined Compression in psi		% Retained Strength
			Before Immers.	After Immers.	
Limestone	None	Original	214	182	85.1
"	1% A	"	219	199	90.8
"	1% B	"	197	191	96.9
			Av. 210	Av. 191	Av. 90.9
Chat	None	Original	133	120	90.2
"	1.5% A	"	112	106	94.5
"	1.5% B	"	141	121	85.8
			Av. 129	Av. 116	Av. 90.2
Limestone	None	1 year	287	237	82.5
"	1% A	"	257	235	91.4
"	1% B	"	228	204	89.4
			Av. 257	Av. 225	Av. 87.8
Chat	None	1 year	268	218	81.4
"	1.5% A	"	282	239	84.7
"	1.5% B	"	289	278	96.2
			Av. 280	Av. 244	Av. 87.4
Limestone	None	2 years	305	290	95.0
"	1% A	"	271	235	86.8
"	1% B	"	263	251	95.4
			Av. 280	Av. 255	Av. 92.4
Chat	None	2 years	295 .	253	85.7
"	1.5% A	"	322	296	92.0
"	1.5% B	"	352	316	89.8
			Av. 323	Av. 288	Av. 89.2
Limestone	None	3 years	359	298	83.0
"	1% A	"	285	254	89.0
"	1% B	"	303	269	88.8
			Av. 316	Av. 274	Av. 86.6
Chat	None	3 years	328	255	77.7
"	1.5% A	"	400	318	79.5
"	1.5% B	"	383	328	85.7
			Av. 370	Av. 300	Av. 81.0

Both of these bituminous mixes showed excellent resistance to water action. After four days of immersion at 140 deg F the majority of the test specimens retained approximately 90 percent of their original compressive strength. The asphalt concrete made with limestone exhibited somewhat greater resistance to water action than the chat asphalt, particularly after 2 and 3 years of service.

On the basis of this test it would appear that the presence of the additives has a slight beneficial effect on the percent retained strength of those two mixes. The average retained strength for all specimens without an additive was 84.9 percent; for additive A, 88.7 percent; and for additive B, 91.0 percent.

For the original test specimens, the asphalt concrete containing limestone gave much higher compressive stress values than the chat asphalt concrete. The average compressive stress before immersion for the limestone asphalt specimens was 63 percent greater than for the chat specimens and 65 percent greater after immersion.

After one year of service as a pavement surface, however, this trend reversed. The chat asphalt at that time showed compressive stress values, both before and after immersion, which were 9 percent greater than for the limestone. After two and three years of service this advantage increased to approximately 15 percent.

To summarize the increase in "stability," as determined by the unconfined compressive strength before immersion: the asphalt concrete made with limestone showed an increase over its original strength of 22 percent after the first year, 33 percent after the second year, and 51 percent after the third; while the chat asphalt concrete showed a 117 percent increase after the first year, a 150 percent increase after the second, and a 186 percent increase after the third.

Figure 10 illustrates the variation in the compressive strength of asphalt concrete with respect to its transverse location in the pavement, and to its length of service as a pavement surface. These curves show the results on specimens tested after immersion, but have the same general characteristics as for the samples tested before immersion. Similarly, although the asphalt concrete for both of these sets of curves contained additive B, the data is represented for their respective aggregate types.

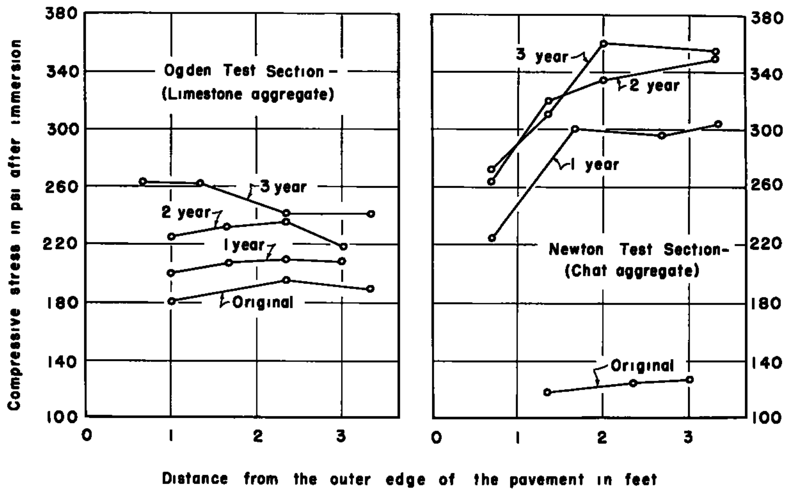

Figure 10. Stability variation in test sections.

The results of the modified immersion-compression test for the beams molded in the laboratory with the roller-compactor were similar to those obtained on the field specimens. The percent retained strengths were slightly higher, possibly due to the fact that the mixes were reheated once in molding the lab specimens. Also, all of the aggregate was coated, whereas in obtaining specimens from the highway, the sawing exposed and cut through the aggregate particles on all four sides of the beams.

During the unconfined compression tests, the flow, or amount of deformation which occurred before the ultimate load was reached, was also measured. Table 6 shows the average flow values for each of the surface types.

E. Sonic Modulus of Elasticity. The sonic modulus of elasticity was determined for both the beams sawed from the pavement and for those molded in the laboratory with the roller compactor. The beams were approximately 16 in. long and 4 in. wide, with the thickness varying from 2.10 to 3.03 in. The fundamental frequencies were determined with the beams at a temperature of 40 deg F, in the manner prescribed by the A. S. T. M. Standards for determining the frequencies of concrete.

The modulus of elasticity was computed from the following expression:

$$E = CWn^2$$

where

E = Young's modulus of elasticity in psi,
C = A factor dependent upon the geometry and the Poisson's ratio of the test specimen,
W = Weight of the specimen in pounds,
n = Fundamental frequency in cps.

TABLE 6

FLOW VALUES FOR THE MODIFIED IMMERSION COMPRESSION TEST

Aggregate Type	Additive	Age	Before Immersion Flow in Inches	After Immersion Flow in Inches
Limestone	None	Original	.234	.264
"	1% A	"	.209	.284
"	1% B	"	.215	.257
			Av. .219	Av. .268
Chat	None	Original	.199	.212
"	1.5% A	"	.246	.253
"	1.5% B	"	.175	.192
			Av. .207	Av. .219
Limestone	None	1 year	.140	.151
"	1% A	"	.135	.182
"	1% B	"	.147	.187
			Av. .141	Av. .173
Chat	None	1 year	.095	.165
"	1.5% A	"	.097	.138
"	1.5% B	"	.086	.122
			Av. .093	Av. .142
Limestone	None	2 year	.129	.186
"	1% A	"	.162	.204
"	1% B	"	.152	.207
			Av. .148	Av. .199
Chat	None	2 year	.111	.130
"	1.5% A	"	.107	.145
"	1.5% B	"	.106	.138
			Av. .108	Av. .138
Limestone	None	3 year	.117	.168
"	1% A	"	.143	.210
"	1% B	"	.128	.210
			Av. .129	Av. .196
Chat	None	3 year	.096	.140
"	1.5% A	"	.098	.124
"	1.5% B	"	.086	.141
			Av. .093	Av. .135

TABLE 7

SONIC MODULUS OF ELASTICITY

		Sonic Modulus at 40 F psi x 106				
		Compacted in	Sawed from the Pavement			
Aggregate	Additive	Lab	Orig.	1 yr.	2 yr.	3 yr.
Limestone	None	1.93	1.81	2.46	2.58	2.87
"	1% A	1.96	1.83	2.56	2.72	2.84
"	1% B	1.91	1.83	2.33	2.45	2.65
		Av. 1.93	1.82	2.45	2.58	2.79
Chat	None	1.15	1.05	1.72	1.82	1.67
"	1.5% A	1.13	.98	1.88	2.02	2.20
"	1.5% B	1.19	1.17	1.87	2.19	2.00
		Av. 1.16	1.07	1.82	2.01	1.96

The averages for the sonic modulus of elasticity for each of the specimen types are shown in Table 7. The asphalt concrete containing limestone again gave higher values than the chat asphalt, particularly for the original specimens. The limestone asphalt sawed from the pavement had a sonic modulus of elasticity 70 percent greater than the chat asphalt before any traffic and aging, and 42 percent greater after three years as a pavement surface.

The presence of the additives had little effect on the sonic modulus. The average for all specimens showed values about 3 percent greater for the samples containing additives.

TABLE 8

ACCELERATED WEATHERING TESTS OF ASPHALT CONCRETE BEAMS

		Compacted in Lab		Sawed from the Pavement Surface					
		(Roller-Compactor)		Original		2 Year		3 Year	
Aggregate	Additive	n^2 [a]	H_2O [b]	n^2	H_2O	n^2	H_2O	n^2	H_2O
Limestone	None	84.2	.63	74.7	.69	78.0	.17	76.4	.35
"	1% A	90.0	.60	83.9	.42	82.2	.19	79.5	.39
"	1% B	89.0	.57	82.5	.38	82.8	.26	76.6	.32
	Av.	87.7	.60	80.4	.50	81.0	.21	77.5	.35
Chat	None	81.4	2.16	73.5	1.69	65.4	1.59	73.8	1.40
"	1.5% A	87.2	1.73	82.5	1.36	79.5	1.22	82.3	1.20
"	1.5% B	83.2	1.74	78.4	1.12	79.3	1.12	82.1	1.13
	Av.	83.9	1.88	78.1	1.39	74.7	1.31	79.4	1.28

a Percent of the resonant frequency squared retained after 5 "weathering" cycles.
b Percent of water absorbed after 5 "weathering" cycles based on the original beam weight.

This was a non-destructive type of test, and after the sonic modulus of elasticity was determined for each beam, it was sawed into quarters for the modified immersion-compression test.

F. Accelerated Weathering Tests. The accelerated weathering test was developed to study the progressive disintegration that occurred in an asphalt beam when subjected to severe "weathering" conditions. The resonant frequency and weight of each beam were determined initially and, for most of the specimens, after one, three, and five weathering cycles, where each cycle consisted of 24 hr in a 140 deg F waterbath, and 24 hr at -15 deg C in air.

With reference to the previous section on the sonic modulus of elasticity, if the geometry and weight of a beam remain constant, E will vary as the square of the resonant frequency (n). Therefore, after a number of weathering cycles, the percent retained n^2 would be approximately equal to the percent retained modulus of elasticity and would be an indication of the resistance of the beam to weathering action. Another measure of the susceptibility of an asphalt beam to weathering action would be the percent of water absorption.

Table 8 lists the percent retained n^2 and percent of water absorption after the beams have been subjected to five weathering cycles. Each value represents the average of the readings for two beams.

In general the asphalt concrete made with limestone showed a slightly higher retained n^2 than the chat asphalt, except for the three year highway specimens. Also, the limestone asphalt concrete, being a much denser mix, absorbed appreciably less water than the chat asphalt.

The accelerated weathering tests showed the additives to their best advantage. For the asphalt concrete made with limestone, the average decrease in n^2 values for specimens containing, respectively, additive A and additive B was only 75 and 80 percent of the n^2 drop which occurred for those samples containing no additives. Similarly, for the chat asphalt concrete the average n^2 drop for additives A and B was only 65 and 73 percent of the decrease for the corresponding no-additive samples.

Figure 11 shows the progressive behavior of the asphalt beams after a varying number of weathering cycles, and illustrates the influence of the additives in decreasing water absorption and increasing the percent of retained E. These two sets of curves are quite representative for both the beams compacted in the laboratory and for those sawed from the pavement.

G. Laboratory Tests on the Asphalt. All of the tests performed on the asphalt itself, both on the original material and on that extracted from the highway, were made by the state highway commission. The extraction procedure, using benzene as the solvent, and the recovery of the bitumen by the modified Abson procedure were carried out in the manner prescribed by the A. S. T. M. standards. Table 9 shows the results of these laboratory tests.

In general the asphalt from the Ogden project showed much better characteristics with time than the asphalt from the Newton test section. The asphalt from this latter project has become quite brittle, as evidenced by the low ductility and penetration values.

The results on the asphalt extracted from the Ogden test section after one year seem inconsistent with the rest of the data, in that the asphalt extracted after two and three years shows much higher ductility and penetration values. It was the opinion of Baldwin and Worley of the state highway commission that the length of time which the beams sat in the laboratory before being tested, contributed to this apparent inconsistency. Due to an oversight, there was about a four month time lapse between the removal of the one-year samples from the highway and the extraction of the asphalt.

H. Surface Roughness Measurements. The initial roughness of each of the test sections was measured with the Highway Commission's Road Roughness Indicator. This single-wheeled trailer is of the standard Bureau of Public Roads design. The roughness values for all the test sections on both projects were between 70 and 80 inches per mile.

There has been very little change in the actual roughness of the test sections after three years of service, particularly on the Ogden project. For the Newton project, the original roughness values for the section containing no additive was 8 percent greater than for the average of the additive sections. After three years, this value has increased to 19 percent.

Additional check runs will be made periodically on these test sections; and the degree of deterioration of the highway surface, expressed as an increase over the original roughness, will be a relative measure of the each section's resistance to traffic and weathering.

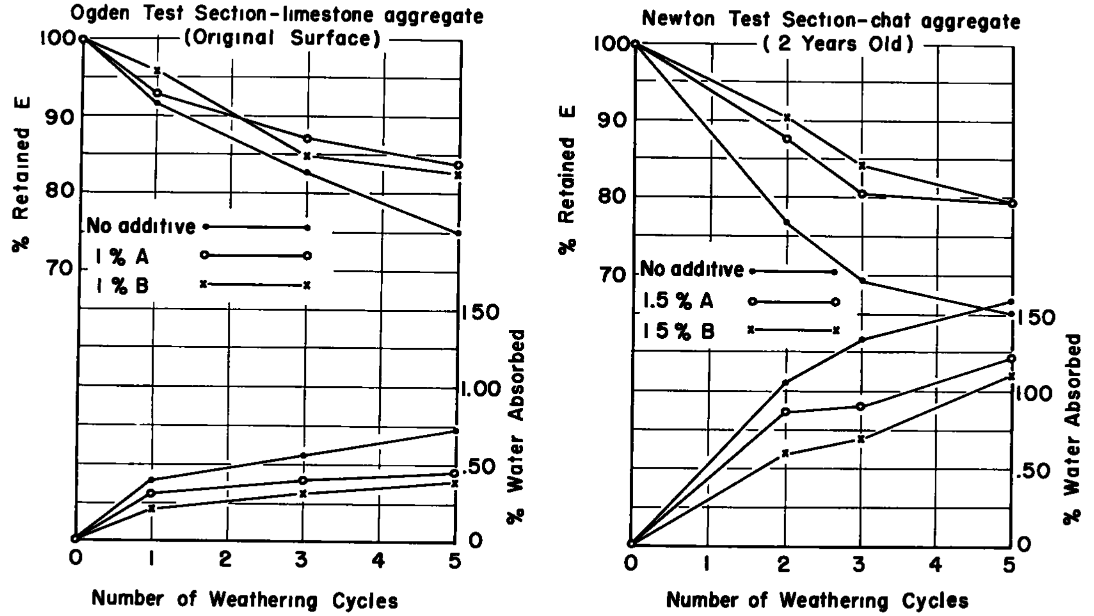

Figure 11. Resistance of test beams to weathering.

TABLE 9

STANDARD TESTS ON THE ASPHALT

	Newton Test Sections (Chat Aggregate)			Ogden Test Sections (Limestone Aggregate)		
	Pen. at 77 F	Softening Point -F	Ductility at 77 F	Pen. at 77 F	Softening Point -F	Ductility at 77 F
Original Asphalt						
No Additive	92	119	100+	92	115	100+
Additive A[a]	108	117	100+	102	113	100+
Additive B	112	116	100+	104	113	100+
Extracted-1 year						
No additive	36	141	13	44	131	57
Additive A	52	131	50	51	129	105
Additive B	38	140	15	45	133	95
Extracted-2 years						
No Additive	33	149	8	63	125	181
Additive A	34	143	9	63	126	176
Additive B	34	147	7	62	125	188
Extracted-3 years						
No Additive	28	153	6	54	129	151
Additive A	32	148	7	60	126	189
Additive B	39	151	6	64	127	193

[a]1½% additive on the Newton test section; 1% additive on the Ogden Test Section.

I. Visual Inspections. In observing the various test sections, there was no appreciable difference noted immediately after laying or after one and two years of service. Similarly, the three-year inspection of the Ogden project indicated no visible signs of distress.

On the Newton test sections, however, during the three-year inspection it was noted that a great many of the coarse aggregate particles had been kicked out of the pavement surface, leaving a pocked-mark appearance. It was the opinion of R.R. Biege, Division Materials Engineer, and the author, that this condition was more noticeable in the sections which contained no additive. The additive sections seemed to have better aggregate retention.

It was also interesting to note that all of the disc markers (refer to Figure 5) remained in place on the additive sections, while none of the markers were retained in the section containing no additive.

CONCLUSIONS

In order to study the behavior of the asphalt concrete as an actual highway surface, the close control with regard to gradation, percent of aggregate, temperature, etc., usually inherent in a laboratory study was sacrificed. This was particularly true for the Ogden project, in which there was a wide variation in the materials of the different test sections. A difference in asphalt content of nearly 1 percent, which did exist between two of these test sections, could easily offset the effect that a small amount of additive might have on the mix. Therefore, the results on the Newton project, which did show good consistency between sections, are probably more significant.

There were a number of other contributing variables in comparing these two test sections: the aggregate types were completely different; the asphalts were both AC-5's, but not from the same source; the capacity, the nature, and the pattern of the traffic differed greatly; and although the design features were essentially the same, the construction practices are seldom identical for two different contractors.

A further complication to the comparison of the different test sections is the fact that both projects are still in excellent condition. Since the difference in properties of the various sections is relatively small, and the number of variables is large, it is a little difficult to isolate the effect of any single one of these many variables.

With these factors in mind, the following summary is presented as an opinion, based on three years of testing, as to the effect of the anti-stripping additives, and a comparison of some of the methods of testing.

A Effect of the Additives. The presence of the additives does retard the hardening of the asphalt, particularly during the first year of service. This is shown in the results of the penetration and ductility tests on the asphalt extracted after one year's time. It is also illustrated in the change in specific gravity of the Newton test sections during the first year; the sections containing the additives experienced a 30 percent greater increase in density than the untreated material.

The effect of the additives in the asphalt is still noticeable after three years as a pavement surface. The values on the extracted asphalt for the Newton project for ductility and penetration were 27 and 8 percent higher, respectively, for the average of the treated samples as compared to the section with no additive. Similarly, after three years the Ogden samples showed a 15 percent ductility and a 26 percent penetration advantage of the treated over the untreated material.

The presence of the additives, both for the original material and after one, two and three years of service, does increase the resistance of the bituminous mixtures to water action. This is probably due in part to the increased density of the treated material. The accelerated weathering test illustrates this property of the additives most markedly, but the modified immersion-compression test also establishes a slight advantage for the treated material.

These test sections will be studied during the entire life of the highway, and subsequent tests may indicate a more definite trend due to the presence of the additives.

B. Methods of Testing. Table 10 is a summary of the "stability" of the original material as determined by three different types of tests on four different types of specimens. The first three specimens were compacted in the laboratory and the last was sawed from the pavement. All four specimens showed much higher stability values for the limestone asphalt concrete than for the chat asphalt, with no definite trend appearing due to the presence of the additives.

The bottom line of Table 10 represents the relative stability of the limestone asphalt concrete compared to that of the chat asphalt, as determined by each of the test methods. There was very close correlation between the unconfined compression tests for the specimens compacted in the laboratory with the roller-compactor and the specimens sawed from the pavement. One test showed the limestone asphalt to be 1.61 times as

TABLE 10

INITIAL "STABILITY" AS DETERMINED BY DIFFERENT TEST METHODS

Aggregate	Additive		Marshall Stability pounds	Immersion-Compression[a] psi	Unconfined Compression[b] Lab Speciman psi	Unconfined Compression[b] From Highway psi
Limestone	None		2,300	757	329	214
"	1% A		2,450	614	331	219
"	1% B		2,380	614	290	197
		Av.	2,380	672	317	210
Chat	None		1,740	265	196	133
"	1½% A		1,480	295	190	112
"	1½% B		1,550	287	208	141
		Av.	1,590	287	198	129
Ratio of Av. Limestone Values / Av. Chat Values			1.50	2.35	1.61	1.63

[a]Unconfined compressive stress before immersion.
[b]Tests performed on blocks approximately 4 x 2½ x 4 .

"stable" as the chat asphalt, while the other showed a 1.63 factor. Both of these values agreed rather closely with the ratio of 1.50, as determined by the Marshall Test.

The resistance to water action and "weathering" of the test sections, as determined by the various test methods, is summarized in Table 11. Once again, the bottom line represents the ratio of the resistance of the limestone sections to the resistance of the chat sections.

TABLE 11

RESISTANCE OF ORIGINAL SPECIMENS TO WATER ACTION AND "WEATHERING"

Aggregate	Additive	Standard Immersion Compression[a] % Ret. Str.	Mod. Immers. - Compr. Molded in Lab % Ret. Str.	Mod. Immers. - Compr. Sawed from Pavement % Ret. Str.	Accelerated Weathering Molded in Lab[b] % Ret. n^2	Accelerated Weathering Sawed from Pavement % Ret. n^2
Limestone	None	68	94	85	84	75
"	1% A	62	94	91	90	84
"	1% B	71	96	97	89	82
		Av. 67	95	91	88	80
Chat	None	90	97	90	81	74
"	1½% A	84	91	95	87	83
"	1½% B	88	93	86	83	78
		Av. 87	94	90	84	78
Ratio of Av. Limestone Values / Av. Chat Values		.77	1.01	1.01	1.05	1.03

a% Ret. Str. - percent retained strength after soaking for 4 days at 140 F.
b% Ret. n^2 - percent retained resonant frequency squared after weathering cycles.

The standard Immersion-Compression test gives a ratio of .77, indicating that the chat asphalt concrete has a much higher resistance to water action than the asphalt concrete containing limestone. The modified immersion-compression test, however, showed ratios of 1.01 for both the specimens compacted in the lab and those sawed from the pavement, indicating the limestone asphalt has slightly better resistance than the chat asphalt. This was substantiated by the accelerated weathering test, in which ratios of 1.05 and 1.03 were determined, respectively, for the lab and field specimens.

It may well be that the 3,000 psi pressure exerted in the molding of a standard immersion-compression specimen is sufficient to fracture some of the relatively weak limestone particles, encouraging the effect of water action. With the hard chat aggregate this would not occur. Nor would this condition occur with either the roller-compactor, in which the maximum pressure does not exceed 100 psi, or in the compaction which a bituminous material receives as a highway surface.

Summarizing, the modified immersion-compression test appears to have some merit as a method of studying the stability and water resistant characteristics of an asphalt concrete pavement. The samples are relatively easy to obtain by sawing them from the pavement surface; and by removing samples at various time intervals, it is possible to determine the change in properties of a mix under the action of time and traffic.

The modified immersion-compression test also seems well suited for a laboratory study when used in conjunction with the roller-compactor. Although the actual stability values were somewhat higher for the lab specimens than for the original field specimens, due probably to reheating and complete aggregate coating, the relative stabilities between the two types of mixes were nearly identical, as measured by the lab and the field specimens. Similarly, the resistance to water action and "weathering" were the same for the beams rolled in the laboratory as for those sawed from the pavement.

The accelerated weathering test, in combination with the determination of the sonic modulus of elasticity, seems to show promise as a means of comparing the resistance of different types of material to the effects of extreme "weathering." This is a non-destructive type of test and has the advantage of allowing a comparison of the properties of a material after a varying number of cycles with the use of only one test specimen, instead of making a large number of "identical" specimens and testing a few of them to failure at the end of each cycle. This test may be performed on beams rolled in the laboratory or on beams sawed from the pavement.

The accelerated weathering test may be too severe in predicting the behavior of a pavement surface. It did indicate that the additives increased the water-resistant characteristics of an asphalt mix, and time may show if these findings are consistent with the behavior of the highway test sections.

ACKNOWLEDGMENTS

The authors wish to acknowledge the complete cooperation of the state highway commission in this program, particularly that of R. L. Peyton, research engineer. The removal of test specimens from the highway, the patching of the sawed-out sections, and the extraction tests were all performed by personnel of the state highway commission. Initial phases of this study were planned in conjunction with Merritt Royer, presently with the Asphalt Institute.

A large amount of the test equipment and the original phases of this study were made possible through a research grant from the Dewey and Almy Chemical Company. This company is the maker of Darakote, one of the anti-stripping additives tested in this research program.

Service Record Study of Bituminous Concrete and Sand Asphalt Pavements in North Carolina

A. DUKE MORGAN, Chief Research Engineer
North Carolina State Highway and Public Works Commission

● THIS is the second performance report on observed behavior of several test sections of specially designed high-type bituminous-concrete paving mixtures. All mixtures have been used as an overlay over old portland cement concrete pavement.

During construction of the project, samples were taken from each truck load for laboratory analysis. Subsequent samples have been taken every year from the area covered by each truckload.

The important factors in this report are the following: (1) Proper design and field control of the mixture, (2) behavior of three types of commercial aggregates under severe service conditions, (3) tack coats, (4) the effect of traffic on density of the pavement after three years of service, and (5) reflection cracking as related to design.

The data should be helpful in determining satisfactory design and control methods for future asphalt-pavement construction for heavy-duty highways.

DESIGN PROCEDURE

Among the chief factors considered in designing a high-type bituminous-concrete pavement is stability. Stability may be defined as resistance to deformation. The laboratory has found a close association between density of the graded aggregate and stability. Regardless of the type of aggregate, the grading that produces the highest aggregate density will produce a bituminous mixture having a higher stability than can be obtained with less-dense gradings of the same aggregate. The first problem, then, is to determine the blend of available aggregates that will produce maximum density (minimum void content).

In selecting the formula for the test sections the following items were taken into consideration: stability, density, percent of voids for total mix, and percent of voids filled with asphalt (Table 5). All mixtures selected contained between 3 and 5 percent voids in the total mix and had from 70 to 78 percent of the voids in the aggregate filled with asphalt. Only AP-3 asphalt (85-100 penetration) was used in these mixes. Several other grades of asphalts were used as tack coats.

Three types of coarse aggregate were used—crushed granite, crushed siliceous gravel (about 50 percent crushed particles), and uncrushed gravel. The uncrushed gravel was the same coarse aggregate used in the bituminous-concrete resurfacing where the rapid and serious failures took place on US 301 as described in the original report. All sections are 1,000 ft in length. (See Table 1 for job mix formulas.)

At the time these test sections were constructed, it was felt that a direct relationship existed between the density and stability of any bituminous mixture. This relationship had been noticed during the analysis of routine reports made on previously designed mixtures.

Since the test sections were constructed, a detailed comprehensive laboratory study has been made in an effort to determine the true relationship between density of bituminous mixtures and stability (Figures 1 and 2). Results of this investigation were reported in the February 1956, issue of the ASTM Bulletin. The Unconfined Compression Test, A.S.T.M. D 1074-52T, was used to make this study. The double-end plunger method of compaction was used in forming the test specimens.

It has been shown that the stability of a bituminous concrete mixture increases with an increase in density up to the point where additional compactive effort would fracture the aggregate. In roadway construction, it may be assumed that unless the mixture on the road receives compaction comparable to that obtained in the laboratory, full benefit is not being derived from the materials in use. Sufficient compactive effort should be applied on the road to produce a finished mixture with a void content within the range permitted by the design procedure.

CONSTRUCTION PROCEDURE ON SECTIONS 1, 2, AND 3

Two grades of asphalt were placed separately on these experimental test sections as a tack between the surface course material and the underlying old cement-concrete pavement. Cutback asphalt, Grade RC-2, at the rate of 0.117 gal per sq yd was applied to the western lane of the pavement and hot bitumen, Grade AP-3 (85-100 pen.) at the rate of 0.063 gal per sq yd was applied to the eastern side. Identical construction methods were employed throughout the surfacing operations, and one-way traffic was maintained on the east shoulder of the road while paving operations were in progress.

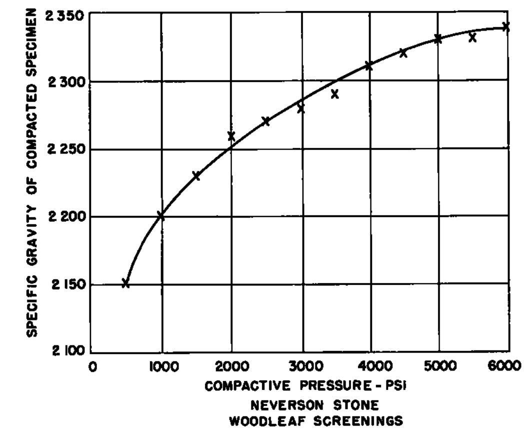

Figure 1.

Each section has a special mix design based on the type of coarse aggregate used. Other materials used in the three designs, such as fine aggregate, mineral filler, and the asphaltic cement, are similar throughout and are from the same source. The rate of application was 150 lb per sq yd.

CONSTRUCTION PROCEDURE ON SECTIONS 4, 5, AND 6

In these three cases, a tack coat of AP-3 (85-100 pen.) asphalt was applied to the western side of the road, at a temperature of 415 F and at the rate of 0.063 gal per sq yd. On the eastern half-width the tack coat consisted of RC-2 and was applied at the rate of 0.120 gal per sq yd. The RC-2 tack coat material was applied two days before the laying of the binder course was begun to allow for sufficient curing.

Each section has a special mix design based on the type of coarse aggregate used. Other materials used in the three designs, such as fine aggregate, mineral filler, and the asphaltic cement, are similar throughout and are from the same source. The resurfacing is composed of 150 lb per sq yd of bituminous concrete binder and 150 lb per sq yd of bituminous concrete wearing surface.

Immediately before spreading the surface-course material a tack coat of Grade AP-3 asphaltic cement, which had a temperature range from 370 to 415 F, was applied to the binder course at an average rate of 0.063 gal per sq yd.

The bituminous concrete was delivered to the paving by trucks loaded uniformly with 6 tons each at an average temperature of 290 F, maximum 390 F, minimum 255 F. The paver was operated at a speed of 18 to 22 ft per minute.

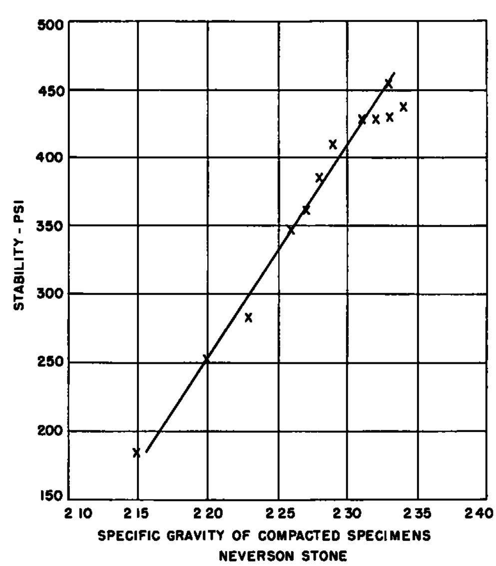

Figure 2.

SECTIONS 7, 8, AND 10

Sections 7, 8, and 10 differ radically from the other test sections in design. Section 7 consists of a standard-specification bituminous surface-treatment mat course which was substituted for the usual hot-mix binder course, and after a 12-day curing period this mat course was surfaced with a laboratory designed hot-plant-mix of bituminous-concrete surface-course material or of hot-plant-mix sand-asphalt surface-course material as prescribed for the subject section. No tack coat was used between the mat and surface courses.

Section 7 was built with a mat-course which consisted of 65 lb per sq yd of specification No. 10 crushed stone (size 1½ in. to No. 4 sieve) and 0.613 gal per sq yd of AE-3 emulsion. The same day this course was applied, 7 lb per sq yd of sand was applied as a blotter course to absorb any excess emulsion. Traffic used this section for 12 days before the surface course was applied consisting of 105 lb per sq yd of bituminous concrete mixture, the same design as the wearing surface on sections 1 and 6.

Section 8 was built with a mat-course of specification No. 10 uncrushed gravel (size 1½ in. to No. 4 sieve) and 0.645 gal per sq yd of AE-3 emulsion. The surface course consisted of 102.6 lb per sq yd of bituminous concrete mixture, the same design as the wearing surface on sections 3 and 4. The construction details of this section are the same as for section 7.

Section 10 was built with a two part mat-course. The first 500 ft consists of 64 lb per sq yd of specification No. 10 uncrushed gravel with 0.589 gal per sq yd of Grade AE-3 emulsified asphalt. The second 500 ft consists of 65 lb per sq yd of specification No. 10 crushed granite with 0.633 gal per sq yd of Grade AE-3 emulsified asphalt. On these mat courses was placed 165 lb per sq yd of hot-plant-mix sand asphalt surface course materials. The same sand was used in all hot-plant mixtures in these test sections. The asphaltic cement used in these surface course mixtures for section 10 consists of AP-5 (60-70 pen.).

OBSERVATIONS OF PERFORMANCE

Section 1: One-Course Bituminous Concrete Containing Crushed Granite Aggregate

The surface of this section is in good condition. No displacement is evident. The reflection of cracks from the underlying old cement-concrete pavement have all appeared in the bituminous concrete surface. This condition had occurred at the end of the first year of service. There is no evidence of instability.

Section 2: One-Course Bituminous Concrete Surface Containing Crushed Gravel Aggregate

Actually the angularity of these aggregates are 49 percent by crushing, 32 percent by nature, and 19 percent of rounded particles.

This section is in very good condition. There is no present evidence of instability. The reflection of cracks from the underlying old cement concrete pavement all appeared in the bituminous concrete overlay by the end of the first year.

Section 3: One-Course Bituminous Concrete Surface Containing Uncrushed Gravel Coarse Aggregate

Some shallow rippling and disturbance of the surface under traffic was evident in this section at the end of the first year. There were some local areas in this section which were characterized by a series of connected threads of asphalt running between the coarse aggregate particles in a direction roughly parallel to the movement of traffic. Such thread patterns have been seen on other roads just before actual distortion of the bituminous concrete takes place. This is construed to indicate that the volume of asphalt or of asphalt and water (total liquid content) in these areas of the pavement is greater than can be accommodated by the voids in the compacted mineral aggregate.

Two years later actual distortion of the bituminous concrete had taken place at these local areas in this section. The analysis of these samples show the specific gravity to be 2.395 which is 101.5 percent of laboratory design density, voids in the total mix to be 0.99 percent and voids filled with asphalt 92.91 percent. The percentage of extracted asphalt is 5.53 or 0.43 greater than the design for this section.

The distortion in this section where most of the distress has occurred is due partly to very poor subgrade under the underlying concrete pavement. This subgrade is characterized as the "pumper type."

Section 4: Two-Course Bituminous Concrete; Uncrushed or Rounded Gravel Coarse Aggregate in Both Binder and Wearing Course

Some displacement and indication of non-uniform asphalt composition are quite evident in the southbound lane of this section. The extracted samples show the average asphalt content in the binder to be 0.63 greater than the laboratory design and the wearing surface to be greater by 0.34.

The northbound lane in this section shows no signs of movement or displacement. The average asphalt content in binder samples is greater by 0.36 than design and the wearing surface less by 0.49 than design. Some slight evidence of raveling appears on the surface which, perhaps, indicated the lack of sufficient asphalt. This would indicate that 4.8 percent asphalt for the surface and 4.4 for the binder is about optimum.

Forty percent of the cracks in the underlying concrete pavement now appear in the surface of the bituminous overlay.

Section 5: Two-Course Bituminous Concrete, Crushed Gravel Aggregate in Both Binder and Wearing Courses, Using the Same Source and Type of Materials as Section 2

The surface of this section is in excellent condition. No displacement is in evidence due to design or construction. The average asphalt content from extracted samples of the wearing surface course is 4.86 percent and from binder samples 4.69 percent. Considering a comparison of the surface area of crushed aggregates versus uncrushed, it is suggested that 5.1 percent should be the optimum asphalt content for the wearing surface and 4.6 for the binder course.

Thirty-eight percent of the cracks in the underlying concrete pavement have appeared in the surface of the overlay.

The increase in roughness after three years services of this section is 16 in. per mi, which compares favorably with section 1 which is 14.5, however, section 1 consists of a wearing surface course only and does not have a binder. (Table 4)

Section 6: Two-Course Bituminous Concrete; Crushed Granite Coarse Aggregate in Both Wearing Surface and Binder Courses

Considerable ruts are discernible in the southbound lane on the low side of the superelevated curve. Asphalt-thread patterns, such as were described in discussing section 3, were seen in many areas of this section. Definite lateral displacement is evident. Some slight lateral displacement occurs in the northbound lane.

The southbound lane of this section was the first pavement laid on these test sections and the portion of this lane in which the greatest amount of displacement has occurred was placed somewhat thicker and with slightly higher asphalt content than designed. These could be contributing factors in this condition.

Thirty percent of the cracks in the underlying pavement now appear in the surface of the overlay pavement.

Section 7: Bituminous Concrete Wearing Course over a Surface Treatment Mat Course; Crushed Granite Coarse Aggregate in Both Courses

The appearance of this section is very good and shows no signs of displacement. Samples removed from this section showed considerable free moisture in the mat course, but whether the uncoated areas seen on the large stone particles is attributable to stripping or to the method of constructing this course is not clear.

The riding quality of this section is very good. There is an increase of only 11.7 in. of roughness per mile during this three-year period.

It seems noteworthy that only 15 percent of the cracks in the underlying concrete pavement are reflected in the bituminous surface overlay.

Section 8: Bituminous Concrete Wearing Course over a Surface Treatment Mat Course; Uncrushed Gravel Coarse Aggregates in Both Courses

The appearance of this section is very good and shows no signs of instability. The riding quality is very good. There is an increase of 10.2 in. of roughness per mile during this three-year period.

As in section 7 only, 15 percent of the cracks in the underlying concrete pavement are reflected in the bituminous surface overlay.

Section 10: Sand-Asphalt Wearing Course over a Surface Treatment Mat Course

The mat course was built in two sections: the first 500 ft consists of crushed granite aggregate and the second 500 ft uncrushed gravel.

The sand asphalt wearing course fine aggregate consists of the same material used in all hot-plant-mix sections.

The laboratory design for the wearing course set the asphalt at 7.5 percent. Road samples show that 7.9 percent was used.

One year after construction 100 percent of laboratory density was obtained.

Slight forward movement or displacement appeared in some areas of the first 500 ft of this (crushed granite) section. Otherwise, the riding quality and general physical condition are good.

Ten percent of the cracks in the underlying concrete pavement are reflected in the bituminous surface overlay. The second 500 ft of this section (uncrushed gravel) shows very little if any movement or displacement. The riding quality and general physical condition are good.

Less than 5 percent of the cracks in the underlying concrete pavement are reflected in the bituminous surface overlay.

Samples have been removed from these test sections annually and each time an attempt was made to evaluate the effectiveness of these tack coats. The values were based on the estimated degree of resistance that the bond between the overlay surfacing and the underlying cement concrete pavement offered when removing the samples.

The degree of bond is expressed in percent: excellent, 100 percent; good, 80 percent; slight, 10 percent; etc.

An over-all average of data after three years shows that for the areas where tack coat material was asphaltic cement, grade AP-3, 36 percent of the original bond was retained. The RC-2 material shows 40 percent of the original bond retained. This indicates that there is little, if any, practical difference between the effectiveness of the two types.

ROAD ROUGHNESS MEASUREMENTS

Roughometer measurements were obtained using a NCSH & PWC roughometer which is patterned after the one-wheel trailor type first built by the Bureau of Public Roads.

The roughness figures reported as units per mile are proportional to the units read from the counter, with the proportionality constant being a factor which is determined by calibration.

Surface roughness measurements were made immediately after construction of these test sections and each year thereafter (Table 4). The original measurements and those taken during the first and second years were made with a high pressure tire. During the third year the high pressure tire was changed to low pressure, which made it necessary to change the standard for evaluating road roughness measurements.

DISCUSSION

It is believed that this project will have a definite effect on future North Carolina specifications for high-type bituminous concrete. It was constructed for a two-fold purpose First, the aggregates used are typical of North Carolina commercial aggregates. The three principal types of coarse aggregates (crushed stone, crushed gravel, and uncrushed gravel) are represented on this project, and the project is considered a test of average commercial aggregates under severe service conditions. Second, it is hoped that the importance of laboratory design and field control will be established through this work.

One of the most important factors in this experiment is the correlation which was found to exist between the laboratory compactive effort for design purposes and the densities obtained from cores taken from the road which show the results of construction rolling.

Bulk specific gravity values are valid only when the pavements compared are alike with respect to specific gravity of the aggregates and mixture composition, including aggregate gradation. Comparisons based on the calculated voids in the compacted mineral aggregate, as it is found in the road surface, would perhaps be more convincing in a demonstration.

After several years of close observation and contact with both laboratory design and field practice in the placing of bituminous plant mix surfaces, it has been repeatedly confirmed that durability and stability of bituminous wearing surfaces are directly related to densities that are obtained during construction.

1. There seems to be no direct relationship between the occurrence of stripping of the asphalt films from aggregate particles and instability of the mixture. Instability occurs without evidence of extensive stripping; on the other hand, considerable stripping is often noted without evidence of instability in the bituminous surfacing. It must be concluded that on this project, there were characteristics of the mixture disposing to success or failure other than the ability of the aggregate particles to retain an asphalt coating.

2. There appears to be no direct relation between the type of coarse aggregate used (stone or gravel) per se, and pavement performance. Both satisfactory and unsatisfactory conditions are noted containing gravel and also crushed stone.

3. There does seem to be a relationship between the range of optimum bitumen contents and the type (stone or gravel) of coarse aggregate.

4. The enriched sections, as shown by sample analysis, evidently are not due to the tack coat applications, since these variations in asphalt content are far greater than the small percentage of asphalt in the tack coat, even assuming that the entire tack coat became a part of the bituminous mixture.

5. There appears to be little if any difference in the effectiveness of AP-3 or RC-2 materials when used as tack coats. There may be a slight advantage in using AP-3 (85-100 pen.) from a construction standpoint, because there would be no waiting period for evaporation which is required when using a cutback such as RC-2.

Careful planning and supervision notwithstanding, there are indications of non-uniform mixture composition within several of the individual sections. By sample analysis it is found actually to exist, and from observation it is shown that the enriched portions of the sections are not giving as good service as the sections in which the asphalt content is near the laboratory design.

ACKNOWLEDGMENT

The author wishes to acknowledge, with sincere appreciation, the help given by all those who have assisted in this experimental work. Special credit should be given to H. D. Hester for accurate records during construction and efficient field sampling, and to H. Fred Waller, Jr., Materials Engineer, for laboratory design work.

Special thanks are extended to C. E. Proudley, Chief Materials Engineer, for the helpful criticism and suggestions in the preparation of this report.

Appendix

TABLE 1

JOB MIX FORMULAS

Passing	Retained	Uncrushed Gravel Percent	Crushed Gravel Percent	Crushed Granite Percent
		Binder Course		
1 in	¾ in	0.82	1.24	0.00
¾ in.	½ in.	23.03	17.88	23.67
½ in.	⅜ in	5.47	13.62	10.54
⅜ in.	No 4	17.83	18.14	14.74
No. 4	No 10	10.09	6.48	7.86
No. 10	No. 40	13.76	14.49	14.07
No. 40	No. 80	14.60	14.14	14.15
No. 200	-	3.26	3.24	3.23
Percent Bitumen	-	4.60	4.40	5.34
		100.00	100.00	100.00
		Wearing Surface		
¾ in.	½ in	0.91	2.60	0.38
½ in	⅜ in.	4.31	18.44	9.35
⅜ in	No 4	28.11	25.04	27.47
No 4	No. 10	18.87	10.86	14.62
No 10	No. 40	14.88	14.04	15.44
No. 40	No. 80	15.60	13.32	15.37
No 80	No. 200	7.29	6.24	7.10
No. 200	-	4.93	4.36	4.47
Percent Bitumen	-	5.10	5.10	5.80
		100.00	100.00	100.00

TABLE 2

COMPARISON OF ANALYSIS OF ROAD SAMPLES WITH DESIGN MIX

	Retained on No. 10 Sieve	Passing No 10 Retained No 200	Passing No 200	Bitumen Percent
	Section No. 1 Surface			
	%	%	%	%
Mix Design	51.82	37.91	4.47	5.80
Test Results	47.44	41.28	5.54	5.74
	Section No 2 Surface			
Mix Design	56.94	33.60	4.36	5.10
Test Results	49.50	39.96	5.47	5.07
	Section No 3 Surface			
Mix Design	52.30	37.77	4.93	5.10
Test Results	55.34	35.14	4.39	5.13
	Section No. 4 Surface			
Mix Design	52.30	37.77	4.93	5.10
Test Results	52.22	37.30	5.41	5.07
	Section No. 4 Binder			
Mix Design	56.42	34.90	3.26	4.60
Test Results	57.18	34.40	3.33	5.09
	Section No. 5 Surface			
Mix Design	56.94	33.60	4.36	5.10
Test Results	47.99	42.28	4.92	4.87
	Section No. 5 Binder			
Mix Design	57.36	35.00	3.24	4.40
Test Results	60.35	31.76	3.21	4.69
	Section No 6 Surface			
Mix Design	51.82	37.91	4.47	5.80
Test Results	49.47	39.35	5.31	5.88
	Section No. 6 Binder			
Mix Design	56.81	36.62	3.23	5.34
Test Results	51.47	39.11	3.93	5.47

TABLE 3

VARIATION IN ASPHALT CONTENT FROM DESIGNED FORMULA

Section No	Asphalt Content, percent Design	High	Low	Ave.	Aggregate Type
1 W.S. W. Lane	5.80	6.17	5.43	5.73	Granite
1 W.S. E. Lane	5.80	6.05	5.14	5.67	"
2 W.S W. Lane	5.10	5.35	4.78	5.02	Crushed Gravel[a]
2 W.S. E. Lane	5.10	5.38	4.78	5.10	" "
3 W.S. W. Lane	5.10	5.53	5.02	5.23	Uncrushed "
3 W.S. E. Lane	5.10	5.29	4.73	5.07	" "
4 W.S. W. Lane	5.10	5.75	5.22	5.44	" "
4 Binder W. Lane	4.60	5.53	5.11	5.23	" "
4 W.S. E. Lane	5.10	4.65	4.55	4.61	" "
4 Binder E. Lane	4.60	5.42	4.81	4.96	" "
5 W.S. W. Lane	5.10	4.87	4.40	4.68	Crushed Gravel[a]
5 Binder W. Lane	4.40	4.77	4.33	4.47	" "
5 W.S. E. Lane	5.10	5.12	4.98	5.05	" "
5 Binder E. Lane	4.40	5.31	4.50	4.92	" "
6 W.S. W. Lane	5.80	6.18	5.75	5.96	Granite
6 Binder W. Lane	5.34	6.00	5.15	5.56	"
6 W.S. E. Lane	5.80	6.15	5.50	5.80	"
6 Binder E. Lane	5.34	6.33	4.88	5.38	"

[a] 49% by crushing, 32% by nature, 19% rounded particles.

TABLE 4

ORIGINAL AND PROGRESSIVE SURFACE ROUGHNESS

Section No.	Original Roughness	First Year	Second Year	Third Year	Total Increase	Aggregate Type
1	71.9	75.5	89.3	86.4	14.5	Crushed Granite
2	64.6	66.8	77.7	80.6	16.0	Crushed Gravel
3	61.7	70.4	102.4	118.4	56.7	Rounded (uncrushed) Gravel
4	53.0	66.8	93.2	96.6	43.6	Rounded (uncrushed) Gravel
5	50.1	51.5	70.4	76.2	16.1	Crushed Gravel
6	57.3	77.1	102.4	106.8	49.5	Crushed Granite
7	67.5	65.3	69.0	79.2	11.7	Crushed Granite Mat.
8	66.1	65.4	85.0	76.3	10.2	Rounded (uncrushed) Gravel Mat.
10	50.6	51.6	66.7	80.9	30.3	Rounded (uncrushed) Gravel Crushed Granite

STANDARD FOR EVALUATING ROAD ROUGHNESS

Roughness Index (Inches per Mile)	Riding Qualities
50 - 60	Excellent
60 - 75	Good
75 - 85	Fair
Above 85	Poor

Note: Roughness measurements are shown in total accumulated inches per mile.

TABLE 5

TABULATION OF AVERAGE VOIDS, VOIDS FILLED WITH BITUMEN AND PERCENT COMPACTION VALUES

	H - F Specimens		Cores 1st Day			Cores-After 1 Yr			Cores-After 3 Yrs		
Wearing Surface Section	% Voids	% Voids Filled with Bitumen	% Voids	% Voids Filled with Bitumen	% Compaction	% Voids	% Voids Filled with Bitumen	% Compaction	% Voids	% Voids Filled with Bitumen	% Compaction
1	2 49	84 29	6 64	65 33	95 7	4 17	74 50	97 9	1 78	88 18	100 8
2	3 28	78 25	6 94	60 97	96 6	6 56	63 10	96 6	2 71	81 28	100 4
3	2 88	80 38	5 74	66 49	97 4	4 11	74 13	98 7	2 14	84 83	100 9
4	3 28	78 25	4 10	74 53	99 1	3 41	76 07	99 1	3 02	79 97	100 0
5	4 07	74 35	4 49	72 77	99 1	3 26	77 85	99 1	2 62	78 63	99 8
6	2 49	84 29	3 72	76 82	99 1	2 92	82 37	99 1	1 83	88 14	100 6
Ave					97 8			98 4			100 4
Binder Section											
4	4 44	70 65	4 10	73 48	98 7	3 33	77 06	97 5	2 82	80 76	100 2
5	2 83	78 53	5 26	65 33	93 3	4 49	70 96	93.3	4 11	72 45	98 2
6	2 47	83 40	4 90	67 95	98 3	3 31	79 50	98 7	2 31	84 64	99 7
Ave					96 8			96 5			99 4

TABLE 6

DESCRIPTION OF MINERAL AGGREGATES

A. Greystone Granite

1. Mineral Classification — Biotite Granite
2. Mineral Composition —
   39% Quartz
   50% Orthoclase
   2% Plagioclase
   6% Biotite
   3% Misc. (Apatite, Magnetitie, Muscovite, Chlorite)
3. Specific Gravity 2.63
4. Absorption 0.4%
5. Los Angeles Wear 39%

B. Lilesville Gravel

1. Mineral Classification — Translucent Quartz and Quartzite Gravel
2. Mineral Composition — Practically Pure Quartz Approximately 99% Silica
3. Specific Gravity 2.64
4. Absorption 0.3%
5. Los Angeles Wear 45%

C. Vander Gravel

1. Mineral Classification — Quartz
2. Mineral Composition — 90%+ Silica and remaining impurities iron oxides
3. Specific Gravity 2.63
4. Absorption 0.6%
5. Los Angeles Wear 39%

TABLE 7

COMPARISON OF LABORATORY AND ROAD SPECIFIC GRAVITIES

Section Number	Design Sp Gr.	Roadway Sp. Gr.	Percentage of Laboratory
		First Day	
1	2.35	2.25	95.7
2	2.36	2.28	96.6
3	2.36	2.30	97.5
4	2.36	2.34	99.2
5	2.36	2.34	99.2
6	2.35	2.33	99.1
Ave.	2.36	2 31	97.9
		First Year	
1	2 35	2 30	97.9
2	2.36	2.28	96.6
3	2 36	2.33	98.7
4	2.36	2.32	98.3
5	2.36	2.34	99.2
6	2.35	2.33	99.1
Ave.	2 36	2.32	98.3
		Third Year	
1	2.35	2.37	100.8
2	2.36	2.37	100.4
3	2.36	2.38	100.9
4	2 36	2.36	100.1
5	2.36	2.35	99.8
6	2.35	2 36	100 6
Ave.	2.36	2.37	100.4

TABLE 8

NORTH CAROLINA SPECIFICATIONS FOR ASPHALT CEMENT AND RAPID CURING CUTBACK ASPHALT

	Grade AP-3	Grade RC-2
Penetration at 77° F	85-100	
Flash Point (Open Cup)°	347+	
Loss on Heating	1.0-	
Penetration of Residue at 77° F		
Percent of original	60+	
Ductility	100+	
Total Bitumen Soluble in CS_2, %	99.5+	
Proportion of Bitumen Soluble in CCL_4, %	99.0+	
Spot Test	Neg. +	
Water, Percent by Volume	0.5-	
Flash Point, Tag, °F	80+	
Viscosity, Saybolt-Furol at 140° F, Sec.	100-200	
Distillation Test:		
Distillate, Percentage by Volume of Total Distillate to 680° F		
To 437° F		40+
To 500° F		65+
To 600° F		87+
Residue from Distillation to 680° F, Percent Volume by Difference		67+
Penetration		80-120
Ductility		100+
Soluble in CCL_4, %		99 5+
Spot Test		Neg. +
Temperature for Application °F		90-160

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