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Materials for Asphaltic Concrete

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Contents

Research on Bituminous Pavements Using the Sand Equivalent Test

R. H. CLOUGH and J. E. MARTINEZ, respectively, Dean, College of Engineering, and Associate Professor, Department of Civil Engineering, University of New Mexico

●THIS PAPER describes a program of research on the sand equivalent test and on the correlation of sand equivalent number with asphaltic concrete performance which was performed at the University of New Mexico under a contract with the New Mexico State Highway Department. To obtain the widest possible range of test values, the Highway Department furnished samples of hot-mix asphaltic concrete and hot-mix aggregate from projects located in every corner of the state.

The first portion of the program involved a detailed study of the sand equivalent test itself. Included were investigations to determine the influence of significant factors such as the type of fine material, amount of fine material, and washing of the aggregate. Also included was an extensive program of testing directed at the correlation of sand equivalent values of an aggregate, before the addition of asphalt and after its extraction.

The second part of the research program was devoted to the study of performance characteristics of asphaltic mixtures and their relationship to the sand equivalent values of the aggregates used for these mixtures. Performance of the asphaltic mixtures was measured in terms of Marshall stability and flow values after simulated weathering by soaking, by the swell characteristics of the mixtures, by the loss in weight caused by abrasion, and by visual examination of the stripping characteristics of the mixes.

USE OF SAND EQUIVALENT TEST

The sand equivalent test was designed to measure, on a volumetric basis, the amount and type of soil fines present in an aggregate in a manner which would indicate the detrimental effect of the fine material on the action of the aggregate under service conditions. The sand equivalent test has the advantages that it is quick to perform, requires only simple equipment which can be used with a minimum of training or experience, and has seemingly yielded good results where it has been adopted as a specification test.

On the basis of his extensive studies on the sand equivalent test, Hveem (2) recommends that specifications prescribe the following minimum sand equivalent numbers:

Materials	Minimum Sand Equivalent Numbers
Crusher run or gravel base materials	30
Aggregates and selected materials for road mix bituminous treatment	35
Aggregates for plant mix bituminous surface	45
Aggregates for asphaltic concrete or Class A plant mix	55
Concrete sand	80

The Arizona Highway Department evaluated the sand equivalent test on the basis of their own test methods and specifications; that is, screen analysis and plasticity index.

Results from tests on a large number of base and mineral aggregate samples were summarized by O'Harra (5), as follows:

Sand Equivalent	Quality of Materials
0 - 14	Unsatisfactory
15 - 24	Doubtful but usually unsatisfactory
25 - 34	Doubtful but usually satisfactory
35 - 54	Almost always satisfactory
55 - 99	Satisfactory

O'Harra also observed that sand equivalent values are affected by factors other than plasticity and gradation. He cited angularity, surface texture, and grain size of the minus No. 200 fraction as factors which can influence sand equivalent numbers. The conclusion reached by the Arizona study was that the sand equivalent test was useful in detecting the presence of excessive amounts of minus No. 200 material.

AGGREGATES USED

The tests of this study were performed on a wide variety of New Mexico aggregates. For convenience in the laboratory work, the aggregates tested and their asphaltic mixtures were divided into four categories. Aggregates 1A through 9A (Table 1) were obtained from the hot bins of paving jobs in progress. Group 1B through 10B (Table 2) were aggregates taken from pits and stockpiles. Aggregates A and B were all separated into fractions and re-combined to satisfy the specification requirements of the particular paving projects where the aggregates were used. Materials 1C through 7C (Table 3) were special aggregates. Number 1C was a concrete sand and Nos. 2C through 7C were soil fines (minus No. 200) of varying activity and fineness which were used in the manufacture of the special D-series aggregates. Aggregates 1D through 12D (Table 4) were artificially manufactured by mixing the same specially prepared coarse aggregate (plus No. 200) with different mixtures and amounts of fines (Nos. 2C through 7C). The specially prepared coarse aggregate was obtained by washing aggregate 2A on the No. 200 sieve, separating the retained material into fractions, and recombining as given in Table 4.

TABLE 1

AGGREGATE MATERIALS TESTED, HOT-BIN AGGREGATES

Ident. No.	Sand Equiv. Value
1A	32
2A	53
3A	34
4A	44
5A	33
6A	31
7A	35
8A	52
9A	56

TABLE 2

AGGREGATE MATERIALS TESTED, STOCKPILE MATERIALS

Ident. No.	Sand Equiv. Value
1B	65
2B	38
3B	34
4B	26
5B	34
6B	33
7B	45
8B	30
9B	30
10B	21

TABLE 3

AGGREGATE MATERIALS TESTED, SPECIAL AGGREGATES

Ident. No.	Description	Sand Equiv. Value
1C	Concrete sand	92
2C	Limestone dust, non-plastic (minus No. 200)	5
3C	Rock flour, high-silica (minus No. 200)	5
4C	Red brick clay (minus No. 200)	0
5C	Bentonite (minus No. 200)	0
6C	Rio Grande silty clay (minus No. 200)	0
7C	Rio Puerco silty clay (minus No. 200)	0

TABLE 4

SPECIALLY MANUFACTURED AGGREGATE[1]

Ident. No.	Soil Fine Agg.	Percent Fines by Wt (-No. 200)	Sand Equiv. Value
1D	4C	6.8	25
2D	5C	5.1	22
3D	2C	8.0	57
4D	3C	6.9	56
	4C	1.1	
5D	3C	7.5	55
	5C	0.5	
6D	6C	8.0	36
7D	7C	8.0	34
8D	3C	8.0	66
9D	3C	4.8	37
	4C	3.2	
10D	3C	5.8	47
	4C	2.2	
11D	3C	6.8	37
	5C	1.2	
12D	3C	7.1	44
	5C	0.9	

[1]Screen analysis (cumulative percent passing by weight: 3/4 in., 100.0; 1/2 in., 94.0; 3/8 in., 86.0; No. 4, 68.0; No. 10, 53.0; No. 40, 30.0; No. 80, 19.0.

LABORATORY PROCEDURES

Sand Equivalent Test

The several hundred sand equivalent determinations made during this study were all performed in accordance with Hveem's "Suggested Method of Test for California Sand Equivalent" (6), and as used by The California State Department of Public Works (3). After dry sieving the aggregate through the No. 4 screen, test samples were obtained by the method of quartering.

Asphaltic Mixtures

Asphalt mixtures 1A through 9A were obtained directly from contractors' pugmills at various New Mexico State Highway projects. It is to be noted again that the designation of asphaltic mixtures uses the same identifying numbers as the aggregates from which they were made. Asphaltic mixes 1B through 10B were prepared in the laboratory using a mechanical mixer. Asphaltic mixtures 1D through 12D were also prepared in the laboratory, a gradation being used which conformed to a U. S. Corps of Engineers specification for dense-graded surface course asphaltic concrete. Aggregates for mixtures 1D through 12D were wetted and thoroughly mixed before heating and adding the asphalt so as to obtain a complete dispersal of fines and to allow them to reach a reasonably natural state of coating on the coarse particles. An 85-100 penetration asphalt cement was used for all asphaltic mixtures, both field and laboratory mixed.

Immersion-Stability Tests

Nine briquettes of each asphaltic mix were molded by the Marshall method using the specified procedure for heating, mixing, and compacting as described in Chapter II, Ref. 4. The briquettes, 2½ in. high by 4 in. in diameter, were compacted by a mechanically driven hammer using 50 blows per side and using a mixture temperature of 275 F ± 5 F. These briquettes were then allowed to cure in air at room temperature for 24 hours. Three specimens from each mix were subjected to the standard Marshall stability test, including immersion in a 140 F water bath for not less than 20 nor more than 30 minutes before testing.

After curing, three briquettes of each mix were immersed in water at 120 F ± 2 F for four days. At the end of this period the specimens were weighed, soaked for 20 to 30 minutes in a 140 F bath, and then tested for Marshall stability. The final three samples of each mixture were immersed in water at room temperature (72 F to 82 F) for 14 days. At the end of this period the specimens were weighed, soaked for 20 to 30 minutes in a 140 F bath, and then tested for Marshall stability. These immersion tests were an adaptation of ASTM: D1075-54 (8), and of the immersion-stability test outlined by Swanberg and Hindermann (7).

All briquettes were weighed in air and weighed while immersed in water one day after molding. Again, after soaking but prior to testing for stability, the specimens were weighed in air in a wet (surface-dry) condition and while suspended in water.

Cold Water Abrasion Test

This test was adapted from information given by Swanberg and Hindermann (7). Eight briquettes, 2 in. in diameter by 2 in. in height, were molded from each mixture at 275 F ± 5 F in Hubbard-Field molds. The quantity of material used for each briquette was computed in advance so that the density obtained would be the same as the average density obtained with the Marshall specimens used in the immersion-stability tests. Each briquette was formed by static compaction to the desired height. Following compaction, the briquettes were allowed to cure in air at room temperature for a period of 24 hours, after which time the eight briquettes for each mix were weighed as a unit.

The briquettes were then immersed in a water bath at 120 F for four days, then weighed again in a wet (surface-dry) condition. The briquettes were then immersed in water at 35 F for one hour after which they were placed in the cylinder of a Deval abrasion machine. The cylinder was filled with water at 35 F and the abrasion test run for 30 minutes at 33⅓ rpm. No steel balls were used, the tumbling of the briquettes themselves providing the only abrasive action. The temperature at the end of the tests ranged from 37 F to 42 F. The eight abraded briquettes of each mix were then weighed as a unit in a wet (surface-dry) condition. The percentage of weight loss due to abrasion was expressed in terms of the original weight.

Swell Tests

The swell tests performed on the asphaltic mixes were conducted in accordance with Method A of AASHO: T101-42 (1).

Stripping Tests

This test was adapted from procedures used by the U. S. Corps of Engineers. About 1, 000 grams of each asphaltic mixture was cured at room temperature for 24 hours. A ½-gallon jar was then filled not more than ⅓ full with the loose, uncompacted mix. The jar was then filled with distilled water and allowed to stand for 24 hours. The jar and contents were then moderately shaken for a 15-min period after which the water was poured off and the sample placed in a pan to dry. A visual qualitative examination was then made of the stripping that had occurred.

DATA AND RESULTS

Sand Equivalent Investigations

An extensive program of tests was conducted to investigate the correlation of sand equivalent values of different aggregates before the addition of asphalt and after its extraction. For this study aggregates were used which displayed a maximum range of sand equivalent values. Samples of each aggregate were split into two fractions. At least six sand equivalent values were obtained by two different operators on the first portion. The second portion of the aggregate was mixed with asphalt, compacted into Marshall briquettes, and cured in air for 24 hours. These briquettes were then heated, broken apart, and the asphalt removed by the use of a solvent in a rotary extractor. Sand equivalent values were then obtained for the separated aggregates.

In studying the variation between the values obtained before and after extraction, many different solvents and combinations of solvents were used. Benzene, methanol alcohol, acetone, ether, chloroform, tri-chlorethylene, detergent, carbon tetrachloride, carbon disulfide, xylene, sodium bicarbonate, and others were used with varying degrees of success in getting the extracted values to check closely with the raw values. No attempt is made to present all of the voluminous data obtained in these studies, but Table 5 gives a representative listing of these values which are of principal interest. Figure 1 also shows this information.

TABLE 5

SAND EQUIVALENT VALUES BEFORE AND AFTER EXTRACTION OF ASPHALT

Aggregate No.	Sand Equiv. Value Before Addition of Asphalt	Sand Equiv. Value After Extraction of Asphalt		
		a	b	c
1A	32	46	38	32
2A	53	60	62	54
3A	34	54	42	38
4A	44	55	52	48
5A	33	49	45	35
6A	31	42	40	32
1B	65	70	68	64
10B	21	42	38	28
1C	92	92	91	92

[a]Extracted with benzene.
[b]Extracted with benzene, rinsed with methanol.
[c]Extracted with benzene, rinsed with acetone and sodium bicarbonate.

The initial extraction tests were performed in accordance with ASTM: D1097-54T (9). As Table 5 shows, the use of benzene solvent yielded a very poor comparison between raw and extracted sand equivalent values. At first it was thought that a possible explanation for this might be the loss of fines which occurs during a rotary extraction, these fines being hurled out of the bowl and retained by the filter paper. To check this, a series of tests was performed on aggregates 1A and 2A in which varying percentages of the minus No. 200 fraction of each aggregate were removed by dry sieving. The results of this investigation are shown in Figure 2. In obtaining the data for the two solid curves in Figure 2, minus No. 200 material was removed by sieving only. It is to be noted that information is also included concerning aggregates 1A and 2A after being washed over the No. 200 sieve.

An additional study of this type was made in which the specially prepared coarse aggregate (plus No. 200) used for the D-aggregate series was mixed with varying amounts of aggregates 2C through 7C (minus No. 200). Figure 3 shows a plot of sand equivalent values versus percent of minus No. 200 fines by weight, in terms of the minus No. 4 material.

Immersion-Stability Tests

A summary of the results of the immersion-stability tests is given in Table 6 and is shown in Figures 4, 5, 6, and 7. Sand equivalent values for each aggregate are compared to the percent of Marshall stability retained after immersion at 120 F for 4 days, and to percent stability retained after immersion at room temperature for 14 days. Comparison is also made between sand equivalent numbers and percent increase in Marshall flow after immersion at 120 F for 4 days, and to precent increase in flow after immersion for 14 days at room temperature.

Data of the asphaltic mix designs used for the immersion-stability tests are given in Table 7. Density, voids, and asphalt content data were computed for each briguette

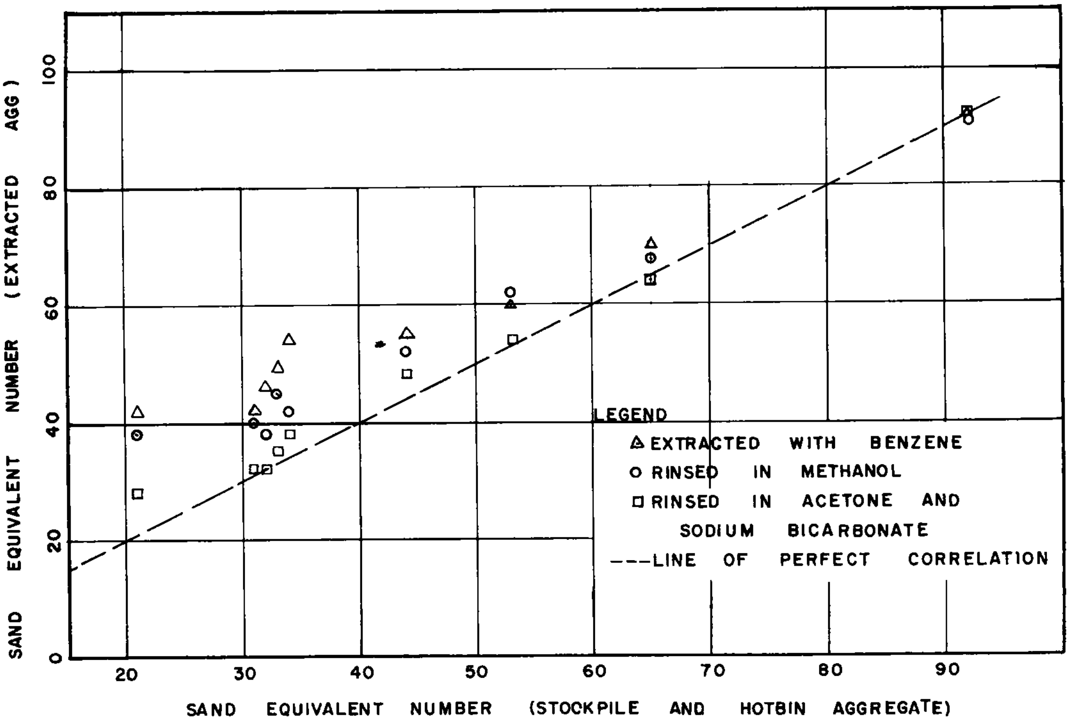

Figure 1. Sand equivalent number of extracted aggregate.

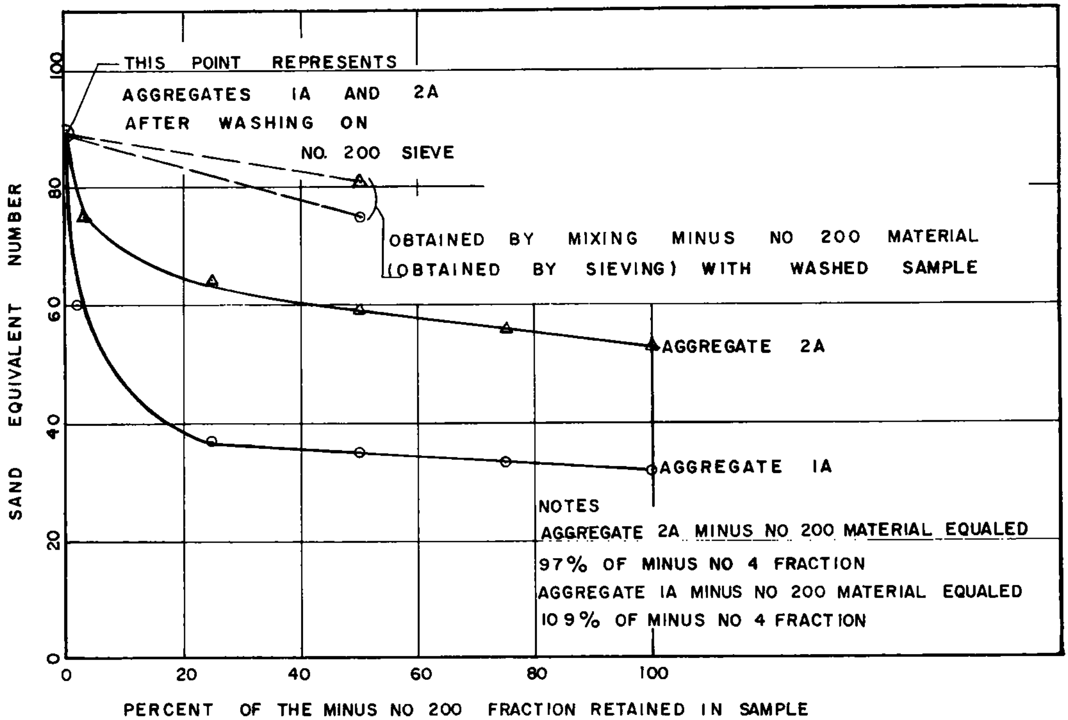

Figure 2. Sand equivalent number compared to percent of minus No. 200 material retained in sample.

and were used for control and comparative purposes. These data are not included, herein, however.

Cold Water Abrasion Tests

Table 8 gives a summary of the results of the cold water abrasion tests. Figure 8 shows a plot of aggregate sand equivalent numbers against percent loss in weight from abrasion. As previously described, the asphaltic mixtures used in these tests were identical with those used in the immersion-stability tests, the only variation being the size of briquettes used.

Swell Tests

Results of the swell tests are given in Table 9. Figure 9 shows these data with sand equivalent number plotted against percent swell. As described under test procedures, the asphaltic mixtures used in the swell tests were the same as those used in the immersion-stability tests.

Stripping Tests

Table 10 gives the results of the stripping tests. Aggregate 2A showed the least amount of stripping of the samples tested. There was a small amount of stripping on the coarse fraction of aggregate 2A but a negligible amount on the fine portion. Aggregate 6A stripped extensively, especially in the minus No. 10 fraction. A considerable amount of stripping occurred in the coarse fraction of aggregate 3A with some of the larger particles being completely stripped. The fines in aggregate 3A were also severely stripped.

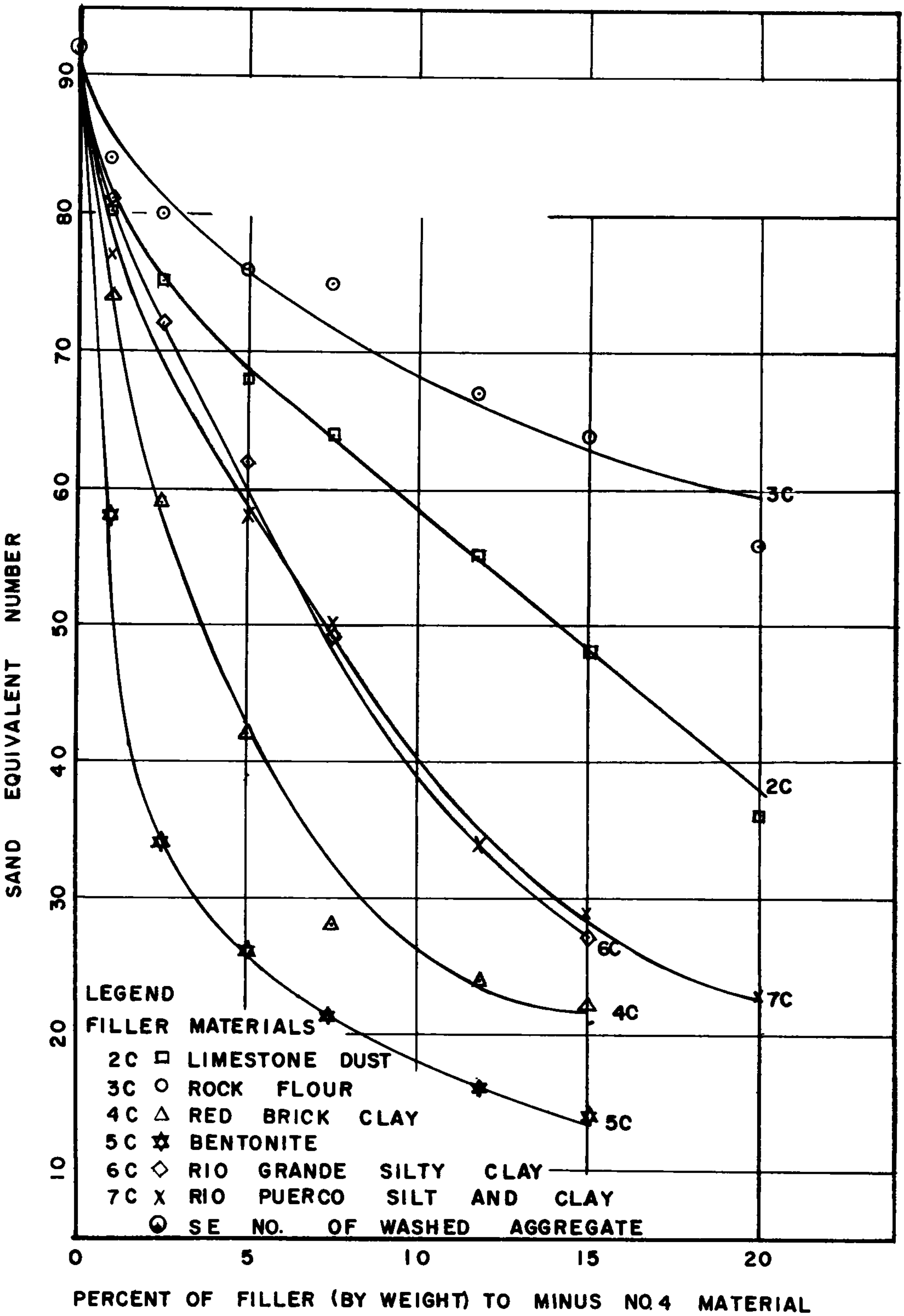

Figure 3. Sand equivalent number for special coarse aggregate (+200) combined with various filler materials (-200).

TABLE 6

SUMMARY OF RESULTS OF IMMERSION, STABILITY TESTS

Sample No.	Sand Equiv. No.	Percent Stability Retained, Soaking Period		Percent Increase in Flow, Soaking Period	
		4 Days	14 Days	4 Days	14 Days
1A	32	51.4	31.9	81.5	129.6
2A	53	100.0	83.4	14.3	25.0
3A	34	87.4	79.6	25.0	37.5
4A	44	69.9	64.0	42.8	21.4
5A	33	79.8	70.2	13.9	19.4
6A	31	47.6	40.6	66.7	60.0
7A	35	57.1	87.4	0.0	8.8
8A	52	84.6	73.1	36.7	40.0
9A	56	91.7	83.7	44.4	55.5
2B	38	71.8	49.2	36.4	66.7
3B	34	82.8	79.7	12.1	6.1
4B	26	53.4	62.1	25.0	37.5
5B	34	86.0	103.5	6.8	2.3
6B	33	65.2	78.3	65.8	81.6
7B	45	91.5	88.6	20.9	11.6
8B	30	77.5	74.8	64.3	54.8
9B	30	82.1	82.3	-2.3	4.6
10B	21	43.3	56.6	68.2	43.2
1D	25	0.0	30.9	-	112.9
2D	22	0.0	17.5	-	139.4
3D	57	81.2	94.9	8.8	35.3
4D	56	72.8	88.7	10.5	5.0
5D	55	96.0	88.2	0.0	0.0
6D	36	57.8	92.6	47.1	11.8
7D	34	72.6	73.6	26.4	10.5
8D	66	91.7	100.1	18.2	-3.0
9D	37	77.0	93.5	46.9	34.4
10D	47	79.6	92.8	14.3	28.6
11D	37	98.0	96.1	-5.4	2.7
12D	44	93.9	89.0	8.3	30.6

DISCUSSION OF RESULTS

Correlation of Sand Equivalent Values

As given in Table 5, the use of benzene alone for asphalt extraction produced a poor correlation between the extracted and raw sand equivalent values. The extracted values were invariably greater than for the original aggregate. The amount of variation between the two values depended on the aggregate, the spread being large for low sand equivalent values and negligible for those aggregates with high values. The use of methanol as a rinse after the benzene extraction was found to improve the correlation, although the spread of values between the raw and extracted aggregates remained substantial. The data obtained using benzene and methanol were used to derive an approximate relationship between extracted and raw values as follows:

$$0.81\ \text{S.E.}_{\text{raw}} + 18 = \text{S.E.}_{\text{extr.}}$$

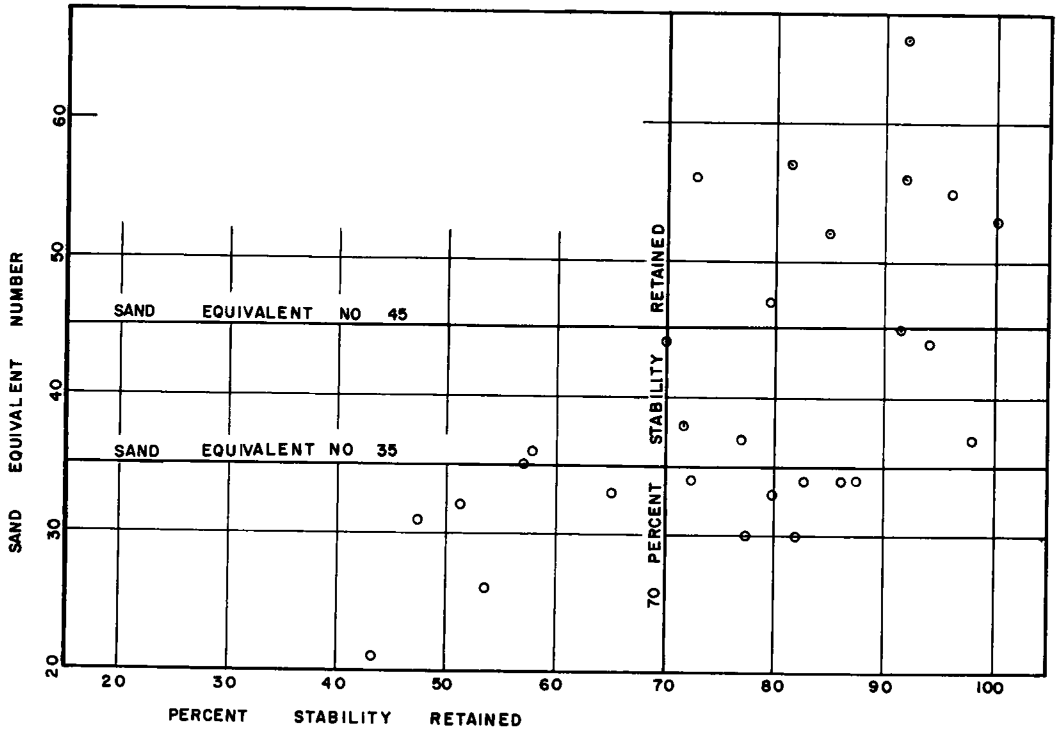

Figure 4. Sand equivalent number of asphaltic concrete mix aggregates compared to percent stability retained after 4-day soaking in water.

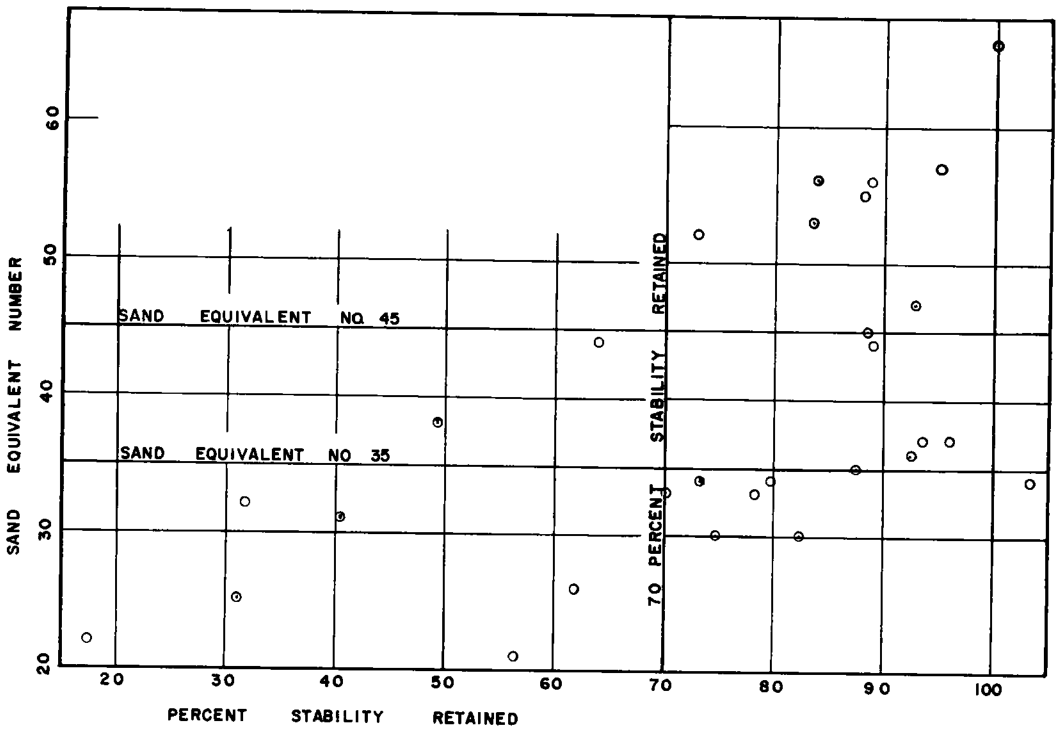

Figure 5. Sand equivalent number of asphaltic concrete mix aggregates compared to percent stability retained after 14-day soaking in water.

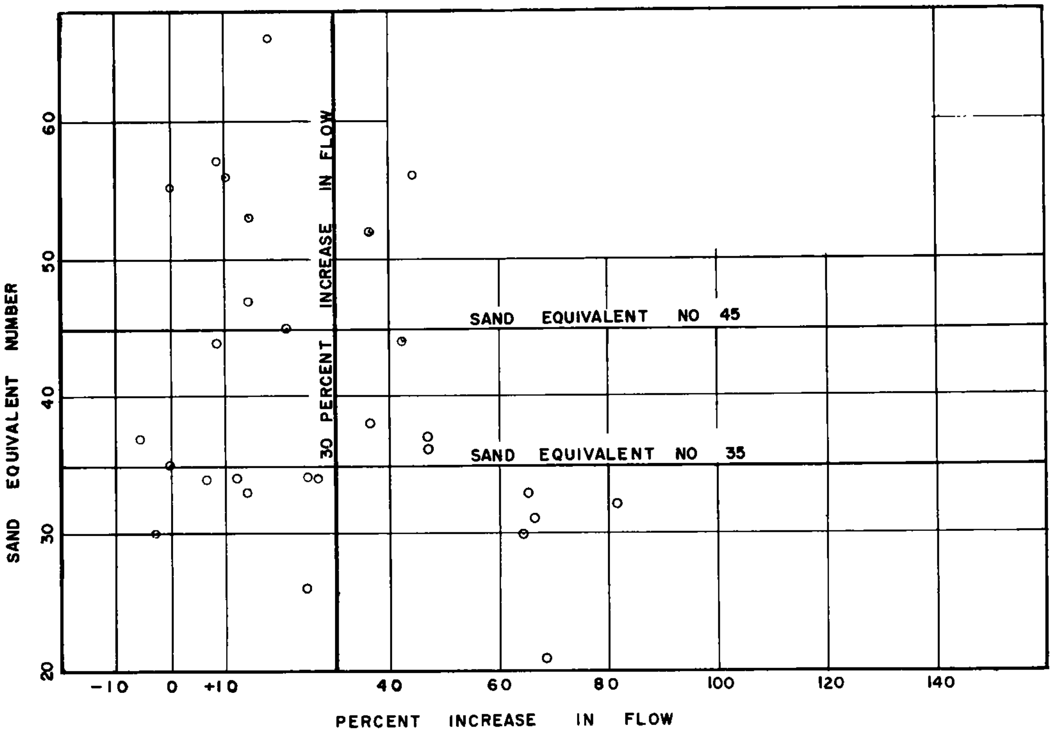

Figure 6. Sand equivalent number of asphaltic concrete mix aggregates compared to percent increase in flow after 4-day soaking in water.

In tests where methanol was used, the aggregate was soaked for about one hour in this chemical which was then removed by rotary extraction. The aggregate was then air dried at room temperature until the alcohol had all evaporated.

As previously mentioned, many chemicals in several combinations were tried in an attempt to secure a better correlation between raw and extracted sand equivalent values. Of these, only the use of acetone and sodium bicarbonate will be discussed because this combination proved to yield the best results. Although the gap between sand equivalent values was never entirely eliminated, extraction with benzene followed by a rinsing with acetone and sodium bicarbonate did achieve a reasonably good check (Table 5). After extraction with benzene, the aggregates were soaked approximately one hour in a mixture of acetone and a water solution of sodium bicarbonate. This solvent was then removed by the rotary extractor, and the aggregate was allowed to air dry at room temperature. It is to be noted that close duplication of results using the benzene, acetone, and sodium bicarbonate proved to be difficult with some scattering of values being in evidence.

Variation of Sand Equivalent Number with Amount of Minus No. 200 Material Present

Because the extracted sand equivalent values were always higher than the original raw numbers, it was believed at first that this might be a result of the loss of fines from the aggregate during the rotary extraction process. This led to the studies summarized in Figures 2 and 3. Figure 2 summarizes the results obtained from a series of tests which determined the sand equivalent values of two aggregates during a process of varying the amounts of minus No. 200 material present.

From Figure 2 it is noted that the increase in sand equivalent value is relatively

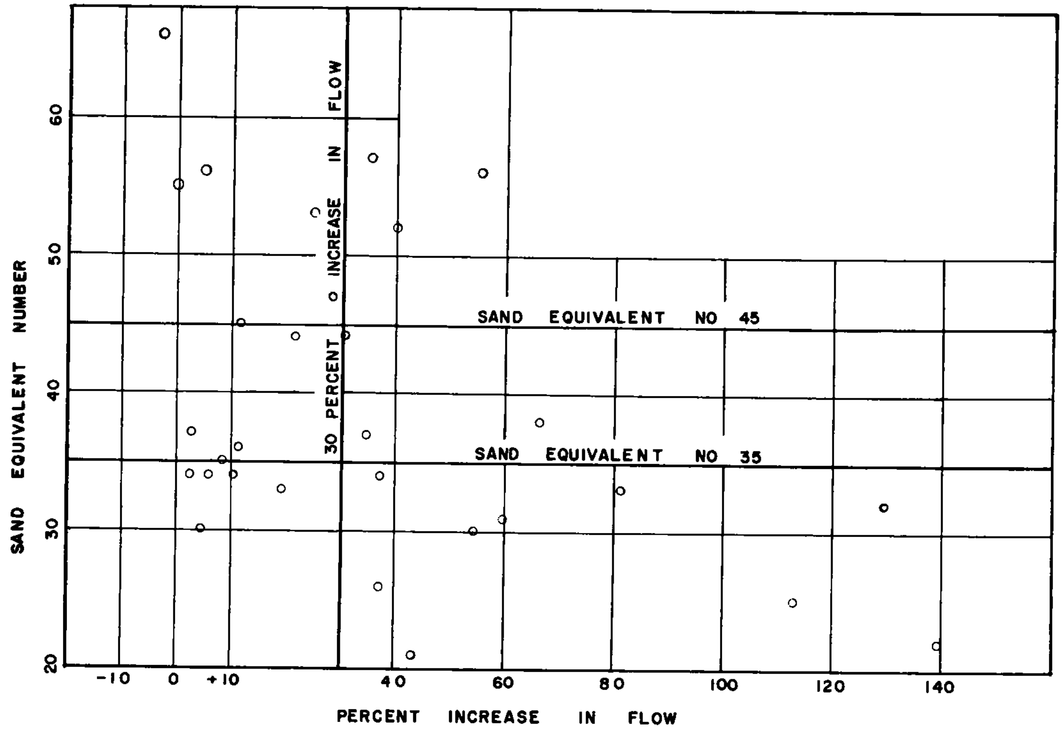

Figure 7. Sand equivalent number of asphaltic concrete mix aggregates compared to percent increase in flow after 14-day soaking in water.

small until more than about 75 percent of the fine material has been removed. Removal of all the minus No. 200 material by sieving alone did produce appreciable increases in the sand equivalent values of both aggregates, although the bulk of this increase accompanied removal of the last 20 percent of the minus No. 200 material. However, a large increase in the sand equivalent value was obtained when the last of the fines was removed by washing. It is interesting to note that aggregate washed over the No. 200 when mixed with 50 percent of its original minus No. 200 material (removed by sieving) had a sand equivalent number considerably higher than the unwashed aggregate which had 50 percent of its original minus No. 200 material removed by sieving.

From these results it appears that the small amount of very fine material which adheres to the larger soil particles exerts a large influence on the sand equivalent values of an aggregate. One practical aspect of this behavior is that the sand equivalent value of an aggregate may be improved only moderately by the process of sieving out the fine material unless substantially all of it can be removed. Washing of the aggregate will yield higher increases in sand equivalent numbers.

The studies summarized by Figure 2 led to the conclusion that the loss of fines during the extraction process was probably not responsible to any degree for the increases in sand equivalent number caused by the addition and extraction of asphalt. Weighing the filter papers before and after extraction showed that the loss of fines did not exceed 5 grams. Considering that the average aggregate sample had approximately 5.5 percent minus No. 200 material (Table 7) and an average asphalt content of 6 percent, the usual 1,000-gram asphalt mixture contained approximately 52 grams of material finer than the No. 200. Thus, at the most, only 10 percent of the minus No. 200 material was lost during the extraction process. Figure 2 would indicate that this loss of fines probably could not account for the differences obtained between the raw and extracted aggregates. These findings resulted in the experimentation with the various other solvents previously listed and discussed.

TABLE 7

ASPHALTIC MIX DESIGN DATA[a]

Sample No.	Percent Passing No. 4	Percent Passing No. 40	Percent Passing No. 200	Percent Asphalt
1A	68.6	38.1	7.5	5.8
2A	64.0	34.7	6.2	6.6
3A	55.6	26.7	5.9	4.4
4A	-	-	-	6.3
5A	57.0	22.1	8.3	5.1
6A	55.8	21.7	5.8	6.0
7A	58.1	26.7	7.4	6.1
8A	67.2	26.7	9.6	7.1
9A	71.8	20.9	6.5	6.1
2B	65.1	27.8	4.7	6.3
3B	60.0	21.9	5.4	5.3
4B	62.3	20.9	5.1	5.8
5B	47.6	18.1	5.3	5.8
6B	53.1	21.1	5.2	6.0
7B	56.0	19.9	6.0	5.8
8B	48.0	19.9	5.2	5.8
9B	52.0	23.6	6.2	6.0
10B	46.0	16.1	5.4	4.5
1D	68.0	30.0	6.8	6.0
2D	68.0	30.0	5.1	6.0
3D	68.0	30.0	8.0	6.0
4D	68.0	30.0	8.0	6.0
5D	68.0	30.0	8.0	6.0
6D	68.0	30.0	8.0	6.0
7D	68.0	30.0	8.0	6.0
8D	68.0	30.0	8.0	6.0
9D	68.0	30.0	8.0	6.0
10D	68.0	30.0	8.0	6.0
11D	68.0	30.0	8.0	6.0
12D	68.0	30.0	8.0	6.0

[a]All aggregates: 100 percent passing 3/4 in.; asphalt: 85-100 penetration A.C.

Influence of Type and Amount of Filler on Sand Equivalent Value

The special coarse aggregate prepared for the D-series aggregates was used as the basic material in this series of sand equivalent tests performed to determine the effect of type and amount of fine material (minus No. 200) on the sand equivalent value. Six different filler materials were used, varying from extremely active bentonite to an inert rock flour consisting of crusher dust. Figure 3 shows the large variation in sand equivalent values of a given coarse material when mixed with different fillers. From Figure 3 it is apparent that the sand equivalent number is not only dependent on the type of fine material, but also on the relative amount present. It will be noted that only about 2 percent, by weight, of bentonite reduced the sand equivalent value to 35, whereas approximately 12 percent of either the Rio Grande or Rio Puerco silty clay was required to accomplish the same reduction. Less than 2 percent of bentonite reduced the sand equivalent to 45, yet it required about 17 percent of limestone dust to produce the same change.

TABLE 8

RESULTS OF COLD WATER ABRASION TEST

Sample No	Weight Before Soaking, g	Weight After Soaking (saturated surface dry), g	Weight After Abrasion (saturated surface dry), g	Loss in Weight (g)	Loss in Weight (%)	Sand Equiv No
1A	1,941 4	2,009 4	1,546 8	462 6	23 02	32
2A	2,009 4	2,023 0	1,905 1	117 9	5 83	53
3A	2,009 4	2,050 2	1,773 5	276 7	13 50	34
4A	2,072 9	2,122 8	1,995 8	127 0	5 98	44
5A	2,068 4	2,122 8	1,927 8	195 0	9 19	33
6A	2,091 1	2,154 6	1,905 1	249 5	11 58	31
7A	2,077 5	2,127 3	1,837 0	290 3	13 64	35
8A	1,982 2	2,045 7	1,914 2	131 5	6 43	52
9A	2,041 2	2,077 5	1,941 4	136 1	6 55	56
2B	1,986 7	2,032 1	1,882 4	149 7	7 37	38
3B	2,041 2	2,059 3	1,905 1	154 2	7 49	34
4B	2,045 7	2,109 2	1,846 1	263 1	12 47	26
5B	2,000 3	2,050 2	1,891 5	158 7	7 74	34
6B	1,905 1	1,991 3	1,642 0	349 3	17 54	33
7B	1,991 3	2,014 0	1,927 8	86 2	4 28	45
8B	1,923 2	1,973 1	1,769 0	204 1	10 34	30
9B	2,018 5	2,059 3	1,927 8	131 5	6 39	30
10B	2,100 1	2,163 6	1,678 3	485 3	22 43	21
3D	2,045 7	2,063 8	1,973 1	90 7	4 39	57
4D	2,059 3	2,086 5	1,977 7	108 8	5 21	56
5D	2,054 8	2,082 0	1,941 4	140 6	6 75	55
6D	2,045 7	2,100 1	1,655 6	444 5	21 17	36
7D	2,050 2	2,104 7	1,850 7	254 0	12 07	34
8D	2,041 2	2,086 5	1,918 7	167 8	8 04	66
9D	2,050 2	2,109 2	1,764 5	344 7	16 34	37
10D	2,059 3	2,104 7	1,823 4	281 3	13 36	47
11D	2,072 9	2,104 7	1,914 2	190 5	9 05	37
12D	2,050 2	2,072 9	1,932 3	140 6	6 78	44

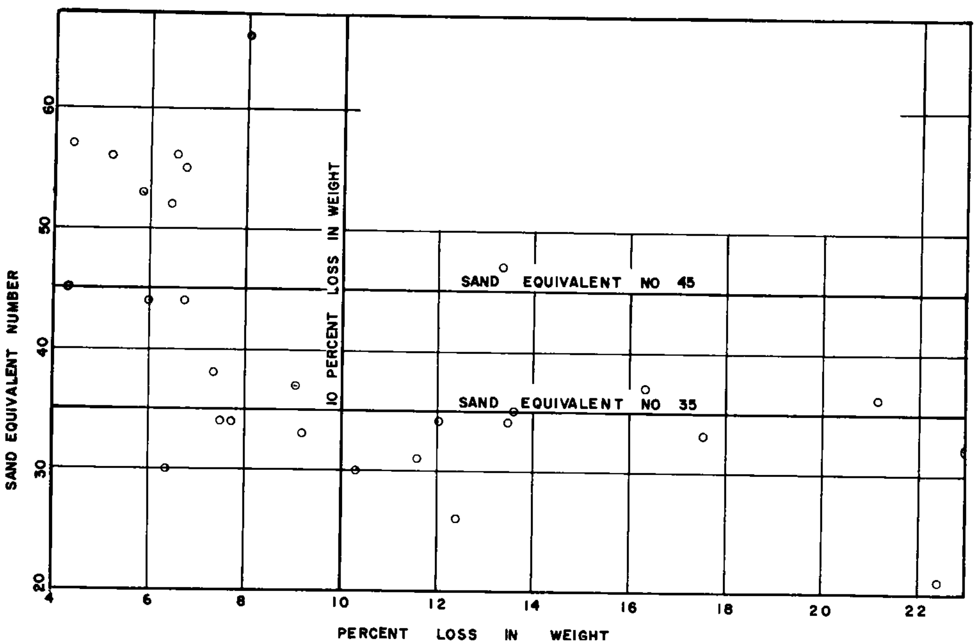

Figure 8. Sand equivalent number compared to percent loss in weight from cold water abrasion test.

Immersion-Stability Tests

The results of the Marshall stability tests performed on the soaked specimens are plotted in Figures 4, 5, 6, and 7. Lines at sand equivalents of 35 and 45 have been drawn on each figure as a basis of comparison with information previously cited (2, 5).

The immersion-stability data, although exhibiting the inevitable scattering of results, do indicate some fairly definite trends. All briquettes prepared with aggregates

TABLE 9

RESULTS OF SWELL TEST
(AASHO: T101-42)

Sample No.	Percent Swell	Sand Equiv. No.
1A	0.18	32
2A	0.09	53
3A	0.36	34
4A	0.09	44
5A	0.05	33
6A	0.59	31
7A	0.00	35
8A	0.23	52
9A	0.14	56
2B	0.18	38
3B	0.00	34
4B	0.27	26
5B	0.13	34
6B	0.26	33
7B	0.04	45
8B	0.22	30
9B	0.13	30
10B	0.28	21
1D	0.09	25
2D	0.78	22
3C	0.00	57
4D	0.00	56
5D	0.10	55
6D	0.00	36
7D	0.00	34
8D	0.05	66
9D	0.00	37
10D	0.00	47
11D	0.00	37
12D	0.00	44

having sand equivalent values of 45 or higher retained a relatively high percentage of stability (70 percent or higher) after being subjected to the action of the water. The retained strength value of 70 percent is used here merely as a guide. A. T. Goldbeck (7) suggests that 70 percent of retained strength in the immersion-stability tests would probably be indicative of satisfactory performance in the field. Only two aggregates with sand equivalent numbers of 35 or higher retained less than 70 percent of stability. More than one-half of the aggregates with sand equivalent values between 30 and 35 retained 70 percent or more of their stability after immersion. No aggregates with numbers below 30 retained as much as 70 percent of their stability.

Although a trend is indicated by the Marshall flow data, it is difficult to establish criteria in terms of flow or percent increase in flow which will act as dividing lines between satisfactory and unsatisfactory performance. A line at 30 percent increase in flow as a result of immersion has been drawn for comparative purposes. From Figures 6 and 7 it may be seen that approximately 25 percent of the asphalt briquettes prepared from aggregates with sand equivalent numbers 45 or higher exhibited more than a 30 percent increase in flow. Similarly, about 50 percent of the aggregates with sand equivalent numbers between 35 and 45 showed a flow increase of 30 percent or

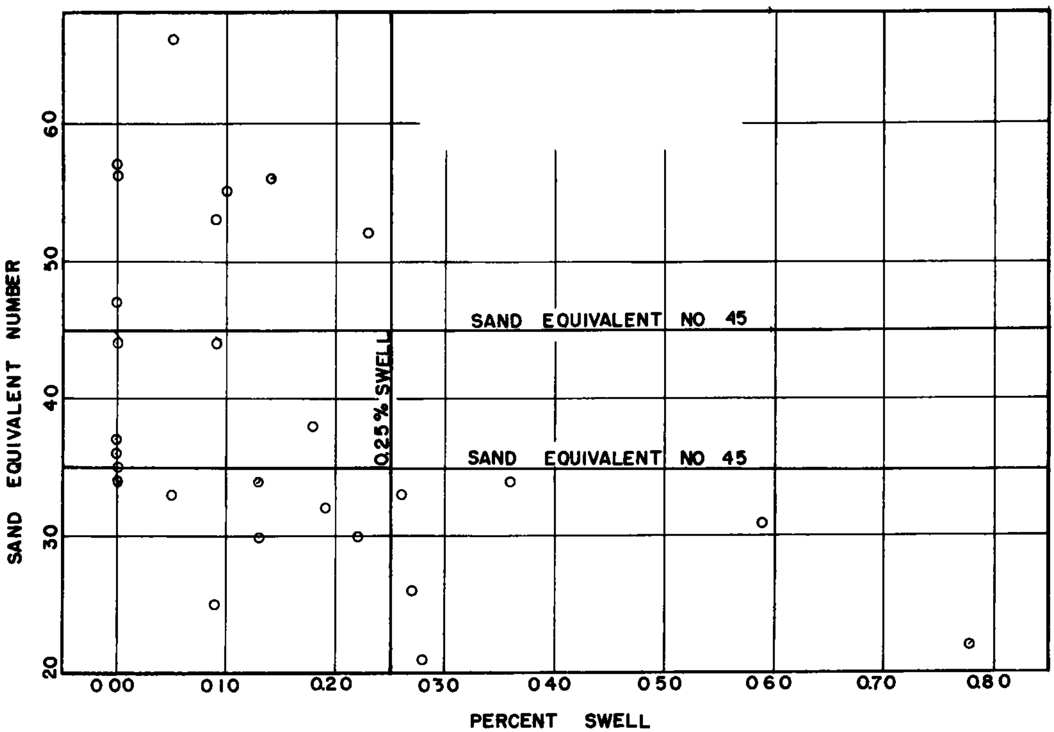

Figure 9. Swell test results (AASHO: T101-42).

TABLE 10

STRIPPING TEST RESULTS

Aggregate (In descending order of resistance to stripping)	Sand Equiv. No.
2A	53
9A	56
4A	44
7A	35
8A	52
5A	33
1A	32
3A	34
6A	31

more, and approximately 55 percent of the aggregates with sand equivalents less than 35 had a flow increase of more than 30 percent. Although no supporting data are available, it is suggested that a maximum increase in Marshall flow of 30 is a reasonable measure of satisfactory performance of an asphaltic mixture with regard to resistance to water effects.

Cold Water Abrasion Tests

The data of the cold water abrasion tests (Fig. 8), illustrate the same general trend as did the data of the immersion-stability tests (Figs. 4, 5, 6, and 7). Both the immersion-stability and the cold water abrasion tests showed that asphalt briquettes made from aggregates with sand equivalent values of 45 or higher possessed good performance characteristics. A line is drawn on Figure 8 at 10 percent loss in weight. This line does not represent a recommended value but is used, rather, for a comparison of test values. A study of Figure 8 shows that the percentage of the asphaltic mixtures showing than a 10 percent loss in weight increases as the sand equivalent values decrease. For sand equivalent values of 45 or higher, about 11 percent of the mixtures exhibited a loss in weight greater than 10 percent. This percentage is approximately 43 percent for sand equivalent numbers between 35 and 45, and about 65 percent for sand equivalent values less than 35.

Swell Tests

The amounts of swell indicated in Figure 9 are small for all mixtures tested. In general, however, asphaltic mixtures prepared from aggregates with higher sand equivalent values exhibited less swell than those from aggregates with lower sand equivalent values. All briquettes whose sand equivalent values were 35 or higher indicated less than 0.25 percent. In Figure 9 a line is drawn along a swell of 0.25 percent for comparative purposes.

Stripping Tests

The results of the stripping tests (Table 10) showed an almost direct correlation between sand equivalent value and resistance to stripping. High values exhibited the most pronounced resistance to stripping. It is recognized that an insufficient number of stripping tests were performed and not enough different aggregates were used to justify any general conclusions. Within the limitations of the data presented, however, the correlation is very pronounced and could have great practical significance. Because of time limitations on the studies, no tests were made on the various aggregates to determine their hydrophobic or hydrophyllic characteristics.

CONCLUSIONS

The data of this program of tests would suggest that Hveem's recommended minimum sand equivalent values for asphaltic mixtures may be somewhat conservative. In combination with gradation and other specification requirements, there is little doubt that Hveem's minimum values will insure the use of high quality materials. However, this study would indicate that adherence to Hveem's values would rule out the use of many New Mexico aggregates whose performance seems to be very acceptable according to these research findings. It is probable that this same situation would be encountered in other areas of the United States.

These studies also provided an extraction process which yielded a reasonably good correlation between sand equivalent values of an aggregate before and after the extraction of asphalt. No studies were made of the correlation of other factors such as plasticity index, but on the basis of a few tests which are not reported herein, it is believed that extraction by benzol and rinsing with acetone and sodium bicarbonate would yield a reasonably good check. This extraction procedure does make it possible to determine from an asphaltic mixture sample a reasonably accurate indication of the sand equivalent value of the aggregate actually used.

ACKNOWLEDGMENT

It is desired to acknowledge the efforts and contributions of John P. Boyd, Delmar E. Calhoun, and James L. Cramer who conducted most of the experimental work and assembled the data reported herein.

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Service Behavior of Asphaltic Concrete

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The design of asphaltic concrete has been based on scientific procedures for the past number of years. The two most used systems of tests, the Hveem stability and the Marshall stability, have both been proved reliable beyond question. Until the validity of the assumptions inherent in the mix design procedures has been proved by sufficient follow-up of service behavior studies and tests, there is danger of overemphasizing stability characteristics at the expense of flexibility characteristics.

Service behavior studies on asphaltic concrete in Oregon were begun in 1954 on a limited scale. In 1959 and 1960 a more extensive survey was organized. One hundred ninety-five paving projects covering 872 miles and representing 3,000,000 tons of asphaltic pavement which have been in service from one to 11 years were surveyed, sampled and tested. The data obtained from the survey, together with the results of routine tests made at the time of construction provide comparative information showing the effect of age and traffic.

The survey consisted of estimating the degree of cracking, raveling, shoving and flushing. Field measurements of wheel track depression and pavement thicknesses were made. Core samples were cut from the area between wheel tracks, in the wheel track and outside the wheel track. An 8- by 12-in. sample was cut from the shoulder or area outside the wheel track.

On arrival at the laboratory the core samples were tested for specific gravity and stability as received, which indicated the in-place condition. The cores were then compacted in the kneading compactor and retested for specific gravity and stability. Granulometric analysis and asphalt content were obtained on a portion of the cut sample. Laboratory-molded specimens, the same size as the cores, were made from the remainder of the cut sample. The molded specimens were compacted in the kneading compactor then inverted and recompacted. Specific gravity and stability were measured after each compaction.

From the analysis of the surface conditions it was found that cracking and raveling is as prevalent as flushing in the pavements surveyed. No evidence was found to indicate that wheel track depressions are caused by instability or horizontal shoving but rather by a combined compaction of pavement and base.

Analysis of the mixture before and after traffic showed a loss of asphalt content after placing in the road.

From the analysis of the results of tests on the cores, it was found that the density increased rapidly for the

first 3 years after construction, then more gradually for the next 8 years. Higher construction compaction requirements would reduce or eliminate the rapid increase by traffic immediately after construction. The density obtained from laboratory-compacted cores changes in a degree depending on the amount of traffic prior to cutting the cores. The greater the prior rolling or traffic, the greater will be the laboratory-compacted density.

From stability tests on the cores, it was found that low in-place values are not indicative of pavement instability, particularly when the mix design is such that further consolidation by traffic increases the measured stability value.

●DESIGN of asphaltic concrete has been based on scientific procedures for the past number of years. Laboratory-molded specimens composed of ingredients to be used for each project are tested to determine the most appropriate gradation and asphalt content. The two most used systems of tests, the Hveem stability and the Marshall stability, have both been proved to be reliable beyond question. Limiting values of stability have been set up for the purpose of insuring stable mixtures under traffic. The assumptions are made that the laboratory-molded specimen is comparable to the actual pavement in regard to particle orientation and density, that the test itself is comparable to traffic loading, and that the limiting value set up distinguishes between good and poor mixtures in reference to shoving and grooving. There are insufficient data available to determine the validity of these assumptions for all types of mixes. The tests and assumptions could well be valid for a sandy mixture but not for a dense but coarser mix such as the Oregon Type B mix. Unless the tests and limiting values are proved valid, there is danger of emphasizing stability at the expense of flexibility.

The presently accepted procedure for increasing the stability of any given asphalt and aggregate mixture is to reduce the asphalt film thickness either by a reduction of asphalt content or by an increase of aggregate surface area. Such a reduction in film thickness will obviously result in a loss of flexibility. If high stability values, as presently determined, are essential to prevent plastic flow under modern heavy traffic, then asphaltic concrete pavement ceases to be flexible and should not be classed as such. Since natural aggregate base materials have been found to retain a characteristic deflection under load regardless of compaction, and asphaltic concrete designed for high stability cannot be used as a wearing surface without inviting failure by cracking and raveling. Treatment of base aggregates for the purpose of reducing deflection under load to the extent that the highly stable semirigid asphaltic mixture can be safely used, reduces the economic advantage asphaltic concrete possesses when used as a flexible pavement.

PURPOSE OF PROJECT

The purpose of this project was to determine the changes in physical characteristics of asphaltic concrete occurring with age and traffic. In addition, the purpose was to determine the degree to which tests on laboratory-molded specimens conform to tests on actual pavement specimens.

SCOPE OF PROJECT

The first series of tests were made in 1954. No research survey project was set up at that time, and the samples were taken and tested whenever time and opportunity presented themselves. Approximately 100 cores were taken from 70 different projects. The results of tests on these samples, rather than providing data for definite conclusions, indicated that further study was needed on a larger scale, both in the number of tests and the pavements represented.

In 1959 the project was set up as a research survey in cooperation with the Bureau of Public Roads with Highway Planning Survey funds sufficient for employment of routine personnel during off-construction season. All the unsealed pavements in Oregon were surveyed, sampled, and tested.

In 1960 the research survey was continued and all the projects on primary highways surveyed the year before, and all those sampled in 1954 were resurveyed, sampled, and tested.

In all, 1,105 cores and 169 cut samples were taken and tested. One hundred ninety-five paving projects were surveyed and sampled. These projects cover 872 miles representing over 3,000,000 tons of asphaltic pavement, and have been in service from one to 11 years under all types of climatic conditions found in the state.

PROCEDURE

No prepared procedure was set up for the 1954 series of samples. A group of projects laid since 1950 were selected at random throughout the state. These projects were examined and sampled by routine personnel as time permitted. One or more cores were cut from the wheel track and the location noted by station or mile post. The wheel track depression was measured, and the general condition of the pavement in the area of test was noted as to indication of shoving, flushing, cracking, or raveling. Laboratory tests consisted of specific gravity and stability on the core samples as received, which indicated the specific gravity and stability in place.

For the 1959 and 1960 surveys, considerable preliminary work was done before the field crew was sent out. The construction test records were examined for each paving project since 1949, and all pavements that had not been sealed were chosen for examination and sampling. The sealed pavements were ruled out for two reasons. First, there is a possibility that the asphalt in the seal coat has penetrated into the pavement and thus both density and stability of the asphaltic concrete would be affected. Second, the visual condition would indicate the performance of the seal coat and not the asphaltic concrete, particularly in regard to flushing or raveling, and possibly shoving.

The test reports on samples taken during construction were scanned for two analyses that were near the average in gradation and asphalt content for the whole project. The points where these two samples were taken during construction were then designated as the points for the survey samples. The station location, granulometric analysis, asphalt content, density after compaction, and other general information were copied to a separate data sheet for each predetermined point. A serial number was assigned to each point consecutively in the field.

When each point was found, the wheel track depression was measured, the degree of flushing estimated on a scale of zero to three, and shoving, raveling, and cracking estimated as none, slight, or excessive. In the 1959 survey, one core was cut in the wheel track and one between the wheel tracks at each point. In the 1960 series, three cores were cut; one between the wheel tracks, one in the wheel track, and one in the outside edge at a point approximately 6 in. toward the wheel track from the edge of the traffic panel. In addition, an 8- by 12-in. cut sample was taken on the outside edge. Each sample was immediately identified with the point serial number and a sample number to correspond with duplicate information on the point data sheet.

On arrival at the laboratory, each core was measured for thickness, exclusive of any asphaltic binder, then cut to 2.5 in. The 2.5-in. sample included the top course of pavement plus any base course needed to make up the height. A series of tests were then made on each core. First, specific gravity and stability were measured. Then, after compaction with the kneading compactor, each core was retested for specific gravity and stability, followed by the determination of real specific gravity. Granulometric analysis and asphalt content were obtained from the cut sample. The remainder of the cut sample was heated and remixed. A molded cylinder was then made by use of the kneading compactor, and the specimen tested for specific gravity and stability. The cylinder was then inverted in the mold and recompacted, after which it was retested for specific gravity and stability.

In the tables and figures which follow, the cores are identified as BWT for between wheel track, WT for wheel track, and SH for the outside edge. Although the laboratory-molded specimens have the same dimensions as the cut core samples, they are identified as laboratory-molded specimens or cylinders and never as cores.

The assumption is made in the test procedure that granulometric analysis and asphalt content are constant transversely across the pavement at the test point, thus making the cut sample and all the cores at a given point comparable. Detailed test procedures are given in the Appendices.

RESULTS OF TESTS

The data obtained from the survey, and tests made on samples taken at the time of construction and at the time of survey provide comparative information showing the effect of age and traffic on some characteristics of asphaltic concrete. Because the surveys were made in the winter, the pavements laid the previous construction season vary in age from three to eight months. The same age differential exists in all the age groups with the effect decreasing to a minimum the eleventh year. An attempt to break down the projects into identical traffic and age groups would be futile, because any two projects laid the same date would not have the same traffic after any given period of time. Also, the difference in climatic conditions makes the effect of traffic and age on one project not comparable with another. In the moderate climate of the coastal area, the pavements seldom have freezing weather on the surface and only occasionally will they absorb enough heat to reach 100 deg. In the midvalleys the range is somewhat greater, whereas in the high plateau area east of the Cascades, the range is from some 40 degrees below zero to a temperature too high to hold one's hand on the pavement. Another condition that makes one project not comparable with another is the difference in aggregate type and gradation so that the same percentage of asphalt will not result in the same film thickness on any two projects.

To diminish, if not remove, the effect of uncomparable individual projects, the data were averaged for the different age groups from zero to 11 years. The data obtained at the time of construction is labeled the original or zero age. The projects that were laid the construction season previous to the survey are indicated as one year, and so on to 11 years. This procedure becomes valid only when the results of analysis portray the effect of age and traffic on pavements as a group, and when no attempt is made to predict the changes that will occur with age on any individual mixture of asphaltic concrete. The only procedure that would make the comparison of test results taken at different ages strictly valid would be to sample and test specific projects yearly. Such a research project is now in progress in which 28 points on six different sections of pavement are observed, sampled, and tested yearly.

The data obtained, pertinent to this report, on pavements one to 11 years of age include nine conditions for determining change from age and for comparison of tests on laboratory-molded specimens and actual pavement specimens. These are as follows:

1. Surface condition—degree of cracking, raveling, shoving, and flushing;
2. Wheel track depression and measured thickness;
3. Gradation and asphalt content—original and final;
4. Density, in-place—original and final in wheel track, final in shoulder and between wheel tracks;
5. Density, laboratory-compacted—final in wheel track, between wheel tracks, and shoulder;
6. Density, laboratory-molded specimen from cut sample—first compaction and second compaction.
7. Stability in-place—original in wheel track for some pavements laid in 1954, final for wheel track, between wheel tracks, and shoulder;
8. Stability laboratory-compacted—final in wheel track, between wheel track, and shoulder; and
9. Stability laboratory-molded specimen from cut sample—first compaction, and second compaction.

TABLE 1
SURFACE CONDITION (%)

Age	Flushing				Raveling			Cracking			Shoving
(yr)	3	2	1	0	Excessive	Slight	None	Excessive	Slight	None	
1	0.0	0.0	0.0	100.0	0.0	0 0	100.0	0 0	0 0	100.0	None
2	0.0	0.0	23 0	77 0	0 0	0 0	100 0	0.0	3 1	96.9	None
3	14 0	10 0	26 0	50 0	0 0	0.0	100 0	0 0	1.7	98 3	None
4	15 5	15 8	32 5	36.2	0 0	16 5	83 5	0 0	28.0	72 0	None
5	0.2	26 5	23 6	49 7	0.0	0 0	100 0	0 0	0 0	100 0	None
6	7.8	10.9	3 9	77 4	5 1	20 3	74 6	0 7	40 5	58 8	None
7	7 0	10 8	54 7	27 5	0 0	9 3	90 7	0 0	43 0	57 0	None
8	0.0	0.0	31 5	68 5	0.0	18 6	81 4	7 8	58 5	33.7	None
9	0.0	0 0	75.0	25 0	0 0	0 0	100.0	0.0	53 5	46.5	None
10	0 0	0 0	77 0	23 0	0.0	18 5	81 5	0 0	32 5	67 5	None
11	0.0	0 0	85 0	15 0	0.0	0 0	100.0	100.0	0.0	0 0	None
Total	6 9	8.1	34 0	51.0	0 9	8 7	90.4	1 7	26 0	72 3	None

Results of the survey concerning surface conditions are given in Table 1. The scale of flushing indicates a visual estimate of four conditions. A rough, dry surface was considered zero. A tight, smooth surface with no free asphalt was considered flushing 1. A slight existence of free asphalt was classed 2. Excess asphalt on the surface was designated as condition 3. The table gives the percentage of mileage classified in each category for each age and for the total of all the projects surveyed. The condition at the point of survey is assumed to represent the mileage for that particular project. The mileage for each condition was added and the percent of the total mileage of the same age was calculated. It should be re-emphasized that the data in Table 1 are group data and should be analyzed as such. For example, of the four-year-old group, 15.5 percent showed excessive flushing and 16.5 percent showed slight raveling. These two conditions are contradictory and would be impossible on the same pavements. The two percentages, however, represent entirely different projects in the four-year-age group. The purpose of this report is to indicate the existing conditions and point out relationships between the different conditions. No attempt is made to isolate the cause of the noted condition.

As indicated in Table 1, there was no visual evidence of shoving in any of the pavements surveyed. There was, however, considerable wheel track depression ranging from zero to 0.75 in. Even though there was no evidence of washboarding or other symptoms of horizontal movement on the surface, the existence of wheel track depressions may be evidence of such movement under the surface. If this were true, then the difference in thickness of between wheel track and shoulder cores and those cut in the wheel track should be comparable to the measured wheel track depression. Table 2 gives the averages of wheel track depressions for the different ages as actually measured and as calculated from thickness and density. The first column shows the averages of measured wheel track. The second column shows the difference between average thicknesses of shoulder and between wheel track cores and the wheel track cores. The third column shows the difference between thicknesses calculated from the average density of between wheel track and shoulder cores and in wheel track cores based on 3.5 in of pavement. The fourth column shows the increase in surface measurements of wheel track depressions measured in 1960 over those measured in 1959.

Comparison of the first three columns of Table 2 indicates that the measured wheel track is consistently greater than that calculated from either the pavement thicknesses or densities. Also, it is seen that the two calculated wheel track depressions are quite comparable, indicating that the decrease in thickness in the wheel track is due to the increased density caused by traffic. The considerably greater actual wheel track depressions indicate that, rather than horizontal instability, there was vertical movement resulting from increased compaction of the base or subbase.

Examination of column one and column four shows a consistency of increasing depression depth in the wheel track from zero to five years age, then decreasing with further traffic and age. A gradual decrease in consolidation of both the pavement and base in the wheel track after a few years, accompanied by a continued consolidation

TABLE 2

WHEEL TRACK DEPRESSION

Pavement Age	Wheel Track Depression (in.) Measured	Calculated from Measured Thickness	Calculated from Densities, 1960 Survey	Increase in Measured, 1959-1960
1	0.07	0.03	0.01	-
2	0.10	0.02	0.02	0.0
3	0.19	0.07	0.04	+0.01
4	0.13	0.03	0.02	+0.01
5	0.31	0.05	0.02	+0.06
6	0.21	0.02	0.04	-0.04
7	0.27	0.03	0.04	-0.02
8	0.20	0.03	0.03	-0.01
9	0.23	0.05	0.01	-0.04
10	0.21	0.03	0.05	-0.05
11	0.17	0.01	0.04	-0.03

between wheel tracks and in the shoulder would account for the measured decrease in wheel track depth after five years.

The gradation and asphalt content, both original and final for each age group, are found in Table 3. Because the original and final asphalt contents are not the same for any of the groups, the percentages in the aggregate portion are calculated to total 100 percent without asphalt. A comparison of the original and final quantity of each size for each age group should indicate the degree of degradation caused by traffic. Examination of Table 3 indicates that there is considerable variation in the change in individual quantities for the different age groups. The quantity passing the number 200 sieve is the quantity which shows an increase with traffic for each age group. Considering the total of 169 specimens of all ages, there is a tendency for the material larger than the number four screen to degrade into material smaller than the number four with the majority being pulverized into dust. This size degradation, however, would appear to be insignificant for asphaltic concrete as a whole.

Of more significance is the drop in quantity of asphalt as indicated for each group in Table 3. The loss of asphalt is not a gradual process since the loss the first year is greater than that indicated in the second, fifth, eighth, and tenth year. Considering the total 169 specimens, 78 percent indicated a loss of asphalt from the time the mixture was placed on the road to the time it was resampled. No relationship can be found between asphalt content or void content at the time of construction and the loss of asphalt. Considering the oldest two groups of pavement, 10 and 11 years, and only those showing a drop in asphalt on retest, the 10-year-old group averaged one-half the asphalt loss and twice the void content as that of the 11-year-old group. This does not preclude a relationship between air or water permeability and asphalt loss because permeability depends on interlocking void space and not on void quantity.

A comparison of the densities in-place at the time of construction with the densities after traffic is shown in Figure 1. In this comparison, core specimens taken from the wheel track only were used in calculating the averages. All the densities are indicated as percent relative compaction which is calculated by dividing the specific gravity of the core as received by the specific gravity after laboratory compaction. Because the majority of cores taken at the time of construction was not laboratory compacted, the compacted value of the wheel track survey cores was used in calculating the original percent relative compaction. The assumption that the specific gravity of a laboratory-compacted core prior to traffic is the same as that after traffic is in error as will be indicated in later comparisons. However, the error applies to all the original specimens which makes them comparable as a group. The average value for percent relative

TABLE 3

GRANULOMETRIC ANALYSIS

Age (yr)	Percent Retained ¾-½	½-¼	¼-No. 4	No. 4-No. 10	No. 10-No 40	No. 40-No. 80	No. 80-No. 200	Pass. No. 200	A.C.
0	8.4	28.4	9 0	12.7	21 0	13 9	3 7	2 9	5.7
1	10.4	32.1	7.9	11 4	19 7	12 8	2.5	3.3	5.1
	+2.0	+3.7	-1.1	-1.3	-1.3	-1 1	-1 2	+0 4	-0.6
0	9 2	30 1	9.7	16.8	17 5	7 3	4.3	5 1	5 6
2	10.9	28.5	8.5	16.2	17 9	7 5	4.5	6.0	5.5
	+1 7	-1.7	-1 2	-0 6	+0.4	+0.2	+0.2	+0 9	-0.1
0	12.4	28 0	7 9	15.8	18.2	7 8	5.1	3 8	5 9
3	14 9	27 8	6.1	15 2	17.6	8.8	5.4	4.8	5 5
	+2.5	-0.8	-1 8	-0.6	-0 6	+1 0	+0 3	+1 0	-0 4
0	7 4	31 4	8 8	17 0	18 5	7.1	4.6	5 2	5 8
4	8 0	32 1	9 0	15 7	17 5	7.0	4 4	6.3	5 2
	+0 6	+0 7	+0 2	-1 3	-1 0	-0.1	-0 2	+1 1	-0 6
0	9 5	33 7	10.2	16 2	14 4	7.1	4 1	4 8	5 8
5	7 6	32.8	10.0	16.8	15.4	7 2	4.5	5 7	5 7
	-1 9	-0.9	-0.2	+0.6	+1 0	+0 1	+0.4	+0.9	-0 1
0	9 1	29.6	10.5	18.7	16 1	5.7	4.2	6 1	6.2
6	10.0	30.9	9.7	17.4	16.3	5.3	3.9	6.5	5 6
	+0.9	+1 3	-0 8	-1 3	+0 2	-0 4	-0.3	+0.4	-0.6
0	12 7	34 4	9.8	16.7	11.7	5.1	5.4	4 2	6.1
7	12 9	36.7	9 4	15 0	10 4	6.2	4.9	4.5	5.3
	+0.2	+2.3	-0.4	-1 7	-1 3	+1.1	-0.5	+0 3	-0.8
0	11 1	32 1	9.9	16.0	17.3	5 1	3.5	5 0	5.8
8	9 1	31.1	9.1	17.0	18.6	6.0	3.5	5.6	5.4
	-2.0	-1.0	-0.8	+1 0	+1 3	+0.9	0 0	+0.6	-0.4
0	16.4	28.8	9.7	15.7	17 1	4 8	3.3	4 2	6.1
9	16.3	29.6	8 7	14.8	16.0	5 5	3.4	5.7	5.5
	-0.1	+0.8	-0.1	-0 9	-1.1	+0.7	+0 1	+1 5	-0.6
0	11.4	27.8	11.0	18.5	16 8	4.9	3.5	5.1	5 4
10	12.3	28.6	10.4	18.2	15 9	4.6	3.4	5 6	5 2
	+0.9	+0.8	-0.6	-0 3	-0 9	-0 3	-0.1	+0.5	-0.2
0	12 2	27.8	11 7	20.0	15 7	3 8	2.7	6.1	4.8
11	13.1	26.2	12 2	20.3	15 2	4.0	2.6	6.4	4 2
	+0.9	-1 6	+0.5	+0.3	-0 5	+0.2	-0 1	+0.3	-0.6
0	11.8	30.3	10.0	16.7	16 2	6 1	3.9	5 0	5 9
Total	11.5	30.9	9 3	16 3	16.3	6 2	3.9	5 6	5 6
	-0.3	+0.6	-0.7	-0.4	+0.1	+0.1	0.0	+0.6	-0.4

compaction at the time of construction indicated in Figure 1 is lower than it would be if laboratory-compacted core values at the time of construction were available for all projects concerned.

The values shown in Figure 1 include the test results of all the wheel track cores cut in the 1954, 1959 and 1960 surveys. Thus, many of the same projects are included in three different age groups. For example, a project constructed in 1953 is included in the one-, six-, and the seven-year groups.

Figure 1 shows the relative compaction as the average, the standard deviation, and the maximum high and low for the original and for each age group. The effect of traffic increases compaction a considerable amount for the first three years after construction, then slightly more for the remaining ages. In calculating the original compactions, the construction values of all the survey projects were averaged. The original average for each group may be either higher or lower than the total average indicated in Figure 1. Therefore, the increase in relative compaction may be for the four-year-age group than for the three-year-age group when the individual original averages are considered. Figure 1 is more a comparison of the percent relative compaction reached than it is a comparison of the amount of increase in compaction for the 11 age groups.

To compare the amounts of increase in compaction, the original relative compactions

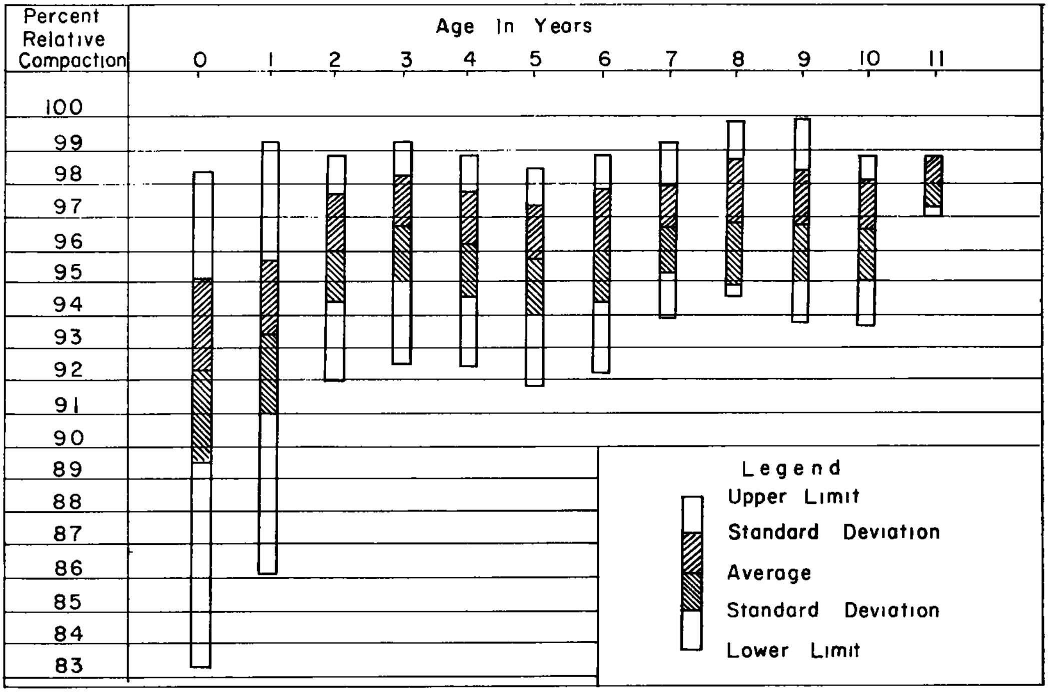

Figure 1. Percent relative compaction with age.

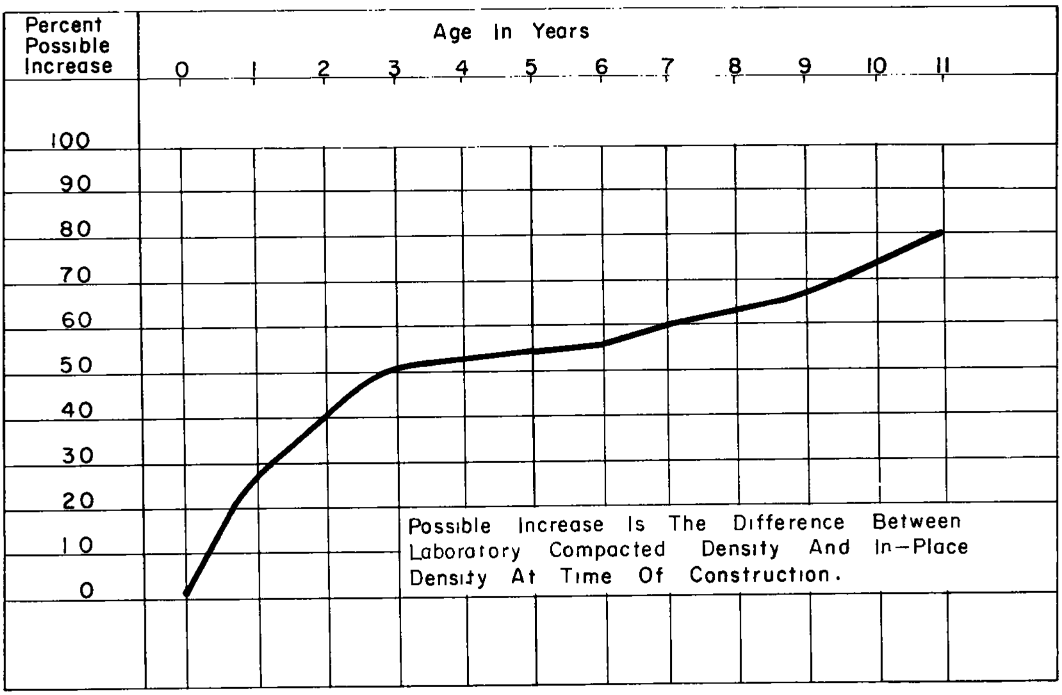

Figure 2. Percent of possible increase in compaction with age.

for all age groups were made comparable by considering the increase as a percent of possible increase. Thus, in Figure 2, the original compaction for each age group is considered zero percent. The difference between the original percent compaction for each age group and the percent compaction obtained after traffic was calculated as percent of the possible increase. It is seen from Figure 2 that approximately 50 percent of the possible increase is obtained in the first three years, and that in the next eight years an additional 30 percent is obtained.

Figures 1 and 2 further indicate that traffic, even after 11 years, does not compact the pavement to the extent that it can be compacted in the laboratory. Because desirable compaction during construction should approach that which traffic will accomplish, so that there will be a minimum of particle movement under traffic, requirement for relative compactions should be based on a laboratory compaction equal to the maximum traffic compaction. It is seen that laboratory compactive effort is higher than it should be.

Table 4 presents a comparison of specific gravities in-place and laboratory-compacted for shoulder, between wheel tracks, and wheel track core samples. Because core specimens were not taken from the shoulder in the 1954 and the 1959 survey, the data presented in Table 4 were obtained from the 1960 survey. The averages for wheel track densities, therefore, will not conform to those presented in Figure 1. Also, the specific gravities of laboratory-molded specimens, made from the cut samples, are shown after both the first and second laboratory compaction. As would be expected, there is an increase in density-in-place values from the shoulder to between wheel tracks and from between wheel tracks to wheel tracks. These are the values used in calculating the wheel track depressions given in Table 2.

The laboratory-compacted cores (Cols. 4, 5, 6, Table 4) show the same pattern of increase in density from shoulder to between wheel tracks to wheel tracks as do the densities-in-place. Although the increase in density with location on the roadway is not so great in the compacted-core data as it is in the in-place data, the definite increase indicates that some change took place with traffic which permits greater laboratory compaction. Further evidence of the effect of traffic on possible compaction is indicated in the last two columns of Table 4. These data were obtained from laboratory-molded specimens the same dimensions as the cut cores and using the mixtures obtained in the cut shoulder samples. After the first compaction, the specimen was tested for specific gravity, then inverted and placed in the mold for recompaction. In each single laboratory compaction, the temperature, number of loads, or tamps, and pressure per tamp were constant. The density obtained from the first compaction of laboratory-molded specimens is consistently lower than the density obtained from compacted wheel track cores. These differences would indicate that the action of traffic on pavement reorients the particles to permit greater laboratory compaction on the core samples, and also, that turning the specimen over permits even greater reorientation and laboratory compaction. Table 5 indicates that this reorientation of particles occurs in the placing and rolling during construction as well as by traffic. Results of tests on 104 cores cut during construction immediately after rolling, along with results of tests on molded specimens of bituminous mixture samples taken prior to rolling at the same points are given in Table 5.

Tables 4 and 5 present data which indicates the difficulty in determining a base density from which to calculate a required relative compaction to be used during construction. The degree to which a pavement can be compacted, as measured by laboratory tests, changes while compaction is being accomplished. It is possible for a contractor, required to obtain further compaction on a project, to operate rolling equipment on the pavement and obtain higher density but still have the same percent relative compaction that was originally considered insufficient. Also, the data indicate that laboratory compaction and ultimate traffic compaction do not result in quite the same densities.

Table 6 presents the average results of Hveem stability tests on all the cores and laboratory-molded specimens obtained in the 1960 survey. Previous laboratory work has indicated a definite relationship between density and stability. This relationship

TABLE 4

SPECIFIC GRAVITIES

Age (yr)	In-Place			Laboratory Compacted			Laboratory-Molded	
	Sh	BWT	WT	Sh	BWT	WT	1st Comp	2nd Comp
1	2 202	2 215	2 225	2.353	2 356	2 362	2.295	2 371
2	2 216	2 248	2 259	2 364	2.370	2 371	2 340	2.409
3	2 307	2 328	2 345	2 413	2 417	2 420	2 373	2 432
4	2 250	2 266	2 278	2 362	2 372	2 374	2 342	2.402
5	2.190	2 255	2 280	2 335	2 365	2 370	2.320	2 370
6	2 260	2.275	2 302	2 391	2.403	2 404	2 344	2 413
7	2 320	2 341	2 370	2 437	2 444	2.450	2 406	2 456
8	2 267	2 293	2 297	2 370	2 375	2 379	2 360	2 402
9	2 227	2 247	2 260	2 307	2 305	2 322	2 265	2 325
10	2 281	2 305	2 318	2 401	2 417	2 431	2 363	2 416
11	2 282	2.320	2 335	2.365	2.377	2 382	2 320	2 372
Total	2 273	2 286	2 319	2 396	2 400	2 406	2 357	2 415

TABLE 5

SPECIFIC GRAVITIES

Job No.	No. of Samples	Cores		Remolded Specimens	
		In-Place	Compacted	1st Compaction	2nd Compaction
1	20	2.22	2.32	2.28	2.35
2	16	2.32	2.45	2.41	2.47
3	4	2.34	2.49	2.48	2.53
4	9	2.26	2.41	2.37	2.43
5	15	2.15	2.29	2.25	2.31
6	6	2.32	2.43	2.43	2.50
7	6	2.14	2.29	2.25	2.32
8	4	2.31	2.52	2.49	2.55
9	10	2.25	2.47	2.46	2.52
10	14	2.21	2.33	2.29	2.53

TABLE 6

STABILITY VALUES

Age (yr)	Group	In-Place			Laboratory Compacted			Laboratory Molded	
		Sh	BWT	WT	Sh	BWT	WT	1st Comp.	2nd Comp
1	A	21 0	20 6	21 8	51 1	51 3	51 5	32 2	49.0
2	A	19 2	20 6	21.4	47 9	49 2	51 0	37 1	48 2
	B	21 3	23 6	23.3	22 0	18 0	31 0	15 6	10.5
3	A	24 6	26 6	21.2	49 0	45 7	47 1	34 2	43 4
	B	22 2	22 3	20 5	27 6	34 1	27 4	18 4	8 5
4	A	22 5	24 5	23 5	45 0	45 0	44 5	33 3	41.0
	B	18 0	17 3	16 3	38 3	40.3	39 0	25 6	12 0
5	A	19 5	22 5	23 0	50 5	52 0	48 0	32 0	46 5
6	A	22 6	21 8	22 0	46 9	39.8	37 9	32 3	45 6
	B	20 1	19 7	20 8	34 4	35 0	33 8	27 2	6 3
7	A	24 1	23.2	24 2	47 0	48 4	43.4	34 7	46.0
	B	19 3	20.4	20.4	22 3	17 7	19 8	15 0	5 5
8	A	22 5	25 8	23.0	48 8	52 8	54 5	39 2	49 2
	B	22 5	26.8	23.5	17 5	24 0	33 5	12 6	0 0
9	A	23 6	29 0	32.3	37 3	49 0	50 0	32 0	36 0
	B	28 0	34.0	32 0	18 0	56 0	25 0	16 0	14 0
10	A	25 7	30.7	28 0	51 0	44 0	41 7	38 0	49 5
	B	15 7	14 5	19 2	4 0	10 7	6 5	11 5	2.7
11	A	22 6	25 6	24 6	44 3	49 0	47 6	38 6	49.0
	B	22 0	26 0	17 0	25 0	39 0	31 0	31 0	22 0

is shown for one mixture in Figure 3 from which it is seen that stability increases to a maximum with increased density, then decreases with further increase in density. Considering the laboratory-molded specimens, it would appear valid to assume that if the stability for the second compaction is less than that for the first compaction, the specimen density is past the peak of its stability-density curve. The data in Table 6 are presented as averages of two groups for each age. In group A the stability for the second compaction is greater than that for the first compaction, and in group B the stability for the second compaction is less than that for the first. The counteracting effect of the two conditions of pavement which would tend to even out averages of the stability-density relationships are thus reduced.

Examination of Table 6 indicates there is some increase in stability of the in-place condition with increase in age from one to 11 years for both groups. In group B it would appear that the relative compaction for the in-place cores for all ages is approaching the peak of the density-stability curve and that further compaction would begin to decrease the stability. In group A, however, compaction would increase to very near 100 percent relative before stability begins to decrease. There are some individual cases scattered through all age groups, total approximately 12 percent of the total, in which the wheel track stability in-place is less than the shoulder and between wheel track stability. These are all in the group B class and the compacted core stabilities and the laboratory-molded specimen stabilities are either zero or approach zero. It would appear that these pavements have reached their maximum stability and that further compaction will decrease stability. Because there was no evidence of instability in the 1960 survey, it is believed that the low stability values in-place are not significant provided the stability-density relationships indicate that the peak of the stability-density curve for the mixture in question has not been reached. The individual specimens that do appear to be on the right slope of the curve in Figure 3 may be susceptible to shoving if further compaction is achieved. However, examination of Figure 1 would indicate that further compaction is doubtful.

The range of individual wheel track stabilities in-place, including the 1954 survey, is from 7 to 38. The average stabilities for group A and for group B for all ages as indicated in Table 6, are presented in Figure 4. This average presentation is quite

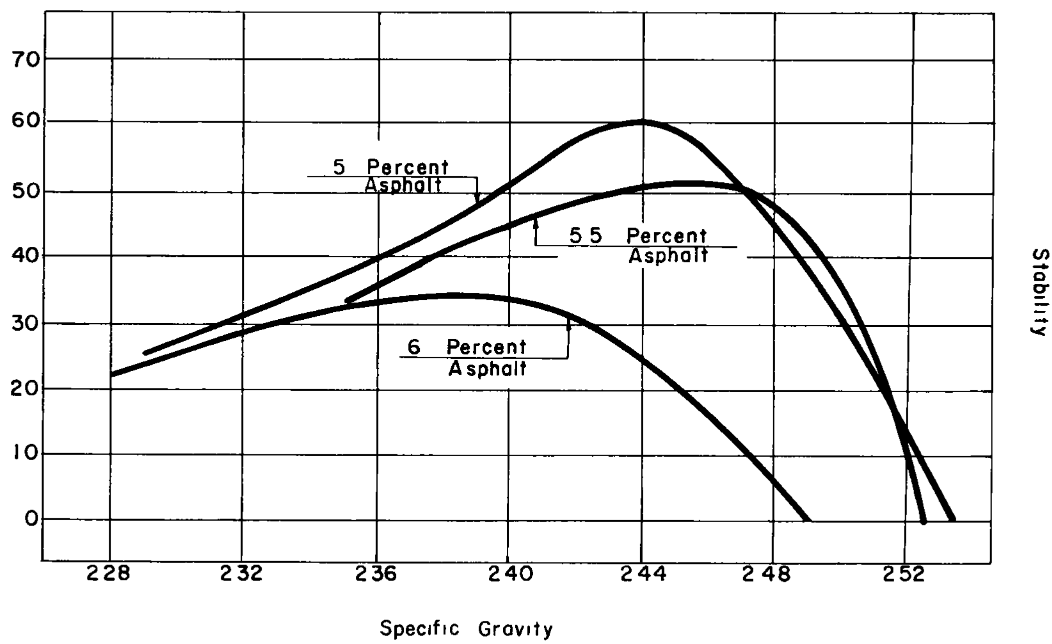

Figure 3. Characteristic stability-density relationship using one aggregate and one gradation.

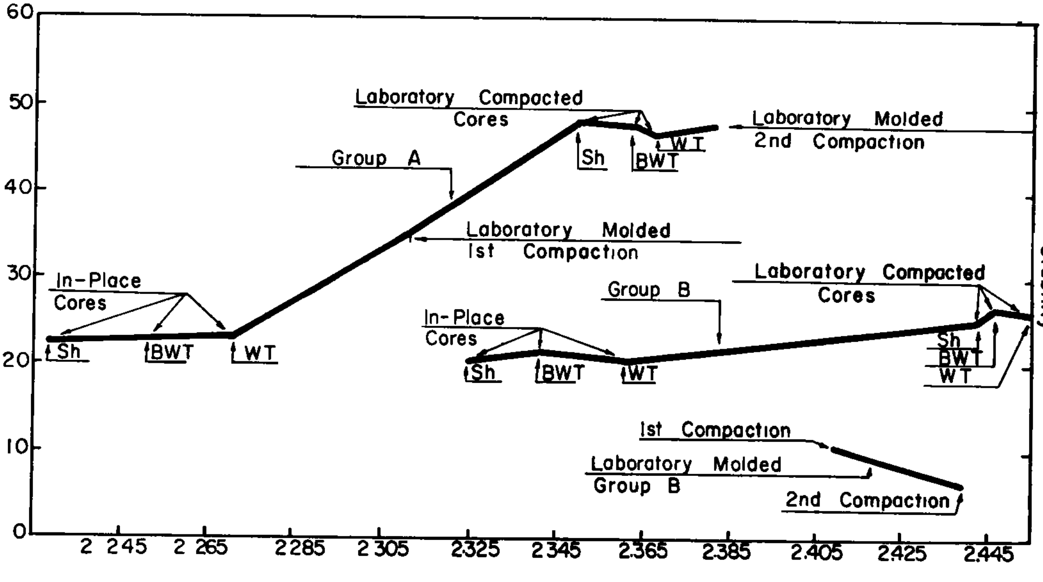

Figure 4. Field condition—laboratory treated.

typical of comparable figures for each age group even though the in-place stabilities are greater for the eleventh year than they are for the first year.

The lowest stability obtained in the 1954 survey was 9 which increased to 17 in the 1960 survey. The average in-place stability in 1954 was 21.4 which increased to 24.8 in 1960.

CONCLUSIONS

From the analysis of the surface conditions obtained from the survey, and tests on samples taken, it is concluded that:

1. Cracking and raveling are as prevalent as flushing in the pavements surveyed, neither of which conditions are excessive.
2. There is no evidence to indicate that wheel track depressions are caused by instability or horizontal shoving of the pavement but rather by a combined compaction of pavement and base.
3. There is a significant loss of asphalt content occurring in many mixtures after they have been placed on the roadway. Both the physical changes taking place and the conditions prevailing which permit such changes remain a matter of conjecture.
4. The rapid increase in density caused by traffic the first three years after construction indicates a need for higher compaction during construction. Traffic compaction continues after construction for at least 11 years, the last eight of which cause only slight particle reorientation. A requirement of 95 percent relative compaction would place the pavement in a condition only slightly susceptible to further traffic compaction.
5. Laboratory compaction on field cut cores is slightly greater than is achieved by 11 years of traffic. The density obtained from laboratory-compacted cores changes in a degree depending on the amount of traffic prior to cutting the cores. The greater the prior rolling or traffic, the greater will be the laboratory compacted value.
6. Low in-place stability values are not indicative of pavement instability, particularly when the mix design is such that further consolidation by traffic increases the measured stability value.

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Appendix A

Equipment and Procedure for Measuring Wheel Track Depressions

The equipment consisted of a 5.5-ft length of 2- by 2-in. aluminum H-beam. One-half inch holes were bored in the flanges close to the web, 2.75 ft from the ends. The holes were bored exactly opposite each other. A ½-in. rod approximately 3.5 in. in length was ground and polished to fit the ½-in. holes with slight clearance. The beam was placed on a level surface with the rod through the holes so that the end of the rod rested on the level surface. The rod was marked at a point even with the top surface of the top flange. An inch of the rod was then calibrated with markings at 1/16-in. intervals from the zero marking toward the upper end.

In making depression measurements, the beam was placed across the wheel track then moved slowly across the roadway until the deepest depression was found which was read on the calibrated rod and recorded.

Procedure for Determining Specific Gravity of Cores and Laboratory-Molded Specimens

Bulk specific gravity determinations were made on core specimens for each condition of compaction. The sample to be measured was weighed in water then wiped surface dry and weighed in air. The sample was then oven dried at 230 F for 15 hours and reweighed. Specific gravity was calculated by dividing the oven-dried weight by the difference between the wet weight in air and the weight in water.

Equipment and Procedure for Determining "Effective" Apparent Specific Gravity for Use in Determining Void Content

The apparatus for this determination consists of a 1-qt fruit jar with a conical cover in which are two vents, one at the peak of the cone and the other part way down the side of the cone. A short metal tube, of approximately 1/16-in. diameter, is threaded into the second vent. The volume of the pycnometer was determined by weight using a solvent of known specific gravity.

In determining the specific gravity of the asphaltic mixture, 500 grams of warm oven-dried sample were placed in the jar and solvent added to near the top. The contents were stirred until the asphaltic mixture had completely disintegrated, and all air bubbles had been removed. The cover was then fastened tight and solvent entered through the tube vent until the top vent overflowed. The jar and contents were placed in a water bath and held for two hours at the temperature used in calibrating the volume. Further solvent was added, if needed, to overflow the pycnometer which was then taken out of the bath, wiped dry, and weighed. The weight of mixture used divided by the volume of the jar minus the volume of solvent resulted in an effective specific gravity value. This value is slightly less than that obtained from the combined specific gravity of asphalt and apparent specific gravity of the aggregate and is slightly higher than the combined specific gravity of the asphalt and the bulk specific gravity of the aggregate.

Equipment and Procedure for Laboratory Compaction and Laboratory-Molded Specimens

The automatic kneading compactor used and the procedure followed is in conformance with ASTM: D 1561-58T. A temperature of 230 F was used for both the consolidation of core samples and the formation of laboratory-molded specimens.

Equipment and Method for Determining Stability

Stabilities of cores and laboratory-molded specimens were determined in accordance with the apparatus and procedure described in ASTM: D1560-58T with the exception that cohesion was not determined.

Appendix B

SEQUENCE OF SAMPLE TREATMENT

Cores

On arrival at the laboratory the core samples were measured for thickness and cut to 2.5 in., then weighed in water, cloth dried and weighed in air. The cores were then placed in tared molds and dried for 15 hr at 230 F and reweighed. This weight is the dry weight used in calculating specific gravity. The molds and cores were brought to 140 F in an oven and the cores transferred to the stability apparatus. After stability measurements were taken the cores were replaced in the molds, heated to 230 F and compacted with the kneading compactor using 160 blows at 450 psi. After compaction, the cores were removed from the molds, weighed in water and in air cloth dried. The dry weight previously used for calculating specific gravity in-place was used for calculating the compacted specific gravity. The cores were then replaced in their molds, heated at 140 F for two hours and retested for stability. The "effective" apparent specific gravity was determined on the cores after the second stability measurements were made.

Cut Samples and Laboratory-Molded Specimens

On arrival at the laboratory the cut samples were divided in half. Granulometric analysis and asphalt content were determined on the top lift of one-half by the rotarex method.

The top lifts of the second half were heated to 140 F for 15 hours, mixed and molded into 2.5-in. specimens with the kneading compactor using 20 blows at 225 psi, then 160 blows at 450 psi and finished with 1,000-lb static load. After removal from the molds the specimens were measured for specific gravity and stability using the same technique as was used for the core samples. The specimens were then inverted in their molds, recompacted using 160 blows at 450 psi and remeasured for specific gravity and stability. These specimens were not measured for "effective" apparent specific gravity.

Pavement Overlays Using Polyester Resin and Asphalt Laminates

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●SELECTED PLASTICS have found wide use in industry because of their high tensile, flexural, and adhesive strength. These qualities, along with resistance to fuel, water, oil, detergents, and oxidation make some of them especially attractive to the highway engineer for pavement coatings.

The major limitations on current overlay systems are high initial cost and the need for precise control and timing during construction. It is now possible to apply resins with controlled setting rates over asphalt concrete (AC) or portland cement concrete (PCC) surfaces (1). With this type of system a substantial improvement in skid resistance, solvent resistance, and durability can be realized compared with conventional coatings and overlays. This paper describes a new overlay method that has these advantages. It is based on ordinary seal coat application methods which simplify construction problems. In addition to asphalt, polyester resin and selected aggregates are used to obtain the desirable qualities needed in the pavement overlay. This new overlay has been demonstrated to be effective by means of several large-scale field tests. The new method is the subject of pending patent application.

DEVELOPMENT OF A LAMINATED SYSTEM

Early experiments by the California Research Corporation used a polyester resin to bond quartz chips directly to a PCC pavement. The resin was sprayed by hand (2) on the surface of a wharf prepared by steam cleaning and a dilute acid etch. After nearly three years of heavily loaded vehicle traffic combined with sea spray, the surface is still intact, although the points of the quartz chips have been dulled. Figure 1 shows one of these surfaces. The cost of this type of coating can be justified where critical conditions exist.

A group of similar experiments was made on an AC main access road in the Richmond Refinery of the Standard Oil Company of California. In this case, these resin surfaces caused the existing AC to crack severely around each experimental section (Fig. 2).

To prevent these cracks around the next test pads, ⅜-in. chips were first bonded by asphalt to the existing AC pavement. The purpose of the asphalt layer was to serve as a slip plane to relieve curing and thermal stresses between the existing pavement and subsequent layers of resin. Moreover, because asphalt is used for the initial bond, resin requirements are less than one-half those of the earlier systems. Precleaning is eliminated for the same reason. In this experiment, a cationic asphalt emulsion was used without precleaning other than sweeping the pavement. Chips were spread into the freshly applied emulsion; and after the emulsion had thoroughly dehydrated, the chips were sprayed with polyester resin. Another layer of ¼-in. chips was then applied, followed by another application of resin, and finally by a layer of sand. There is no sign of cracking in the overlay or in the adjacent AC. It is in excellent condition after nine months of heavy truck traffic. Figure 3 shows this coating, and Figure 4 illustrates a section of typical overlays using this system.

CONSTRUCTION OF LAMINATED OVERLAYS

All of these laminated overlays are constructed in a similar way. The first layer resembles a conventional seal coat with suitable chips or stones being imbedded in an appropriate amount of asphalt. The asphalt rate and aggregate size and type are

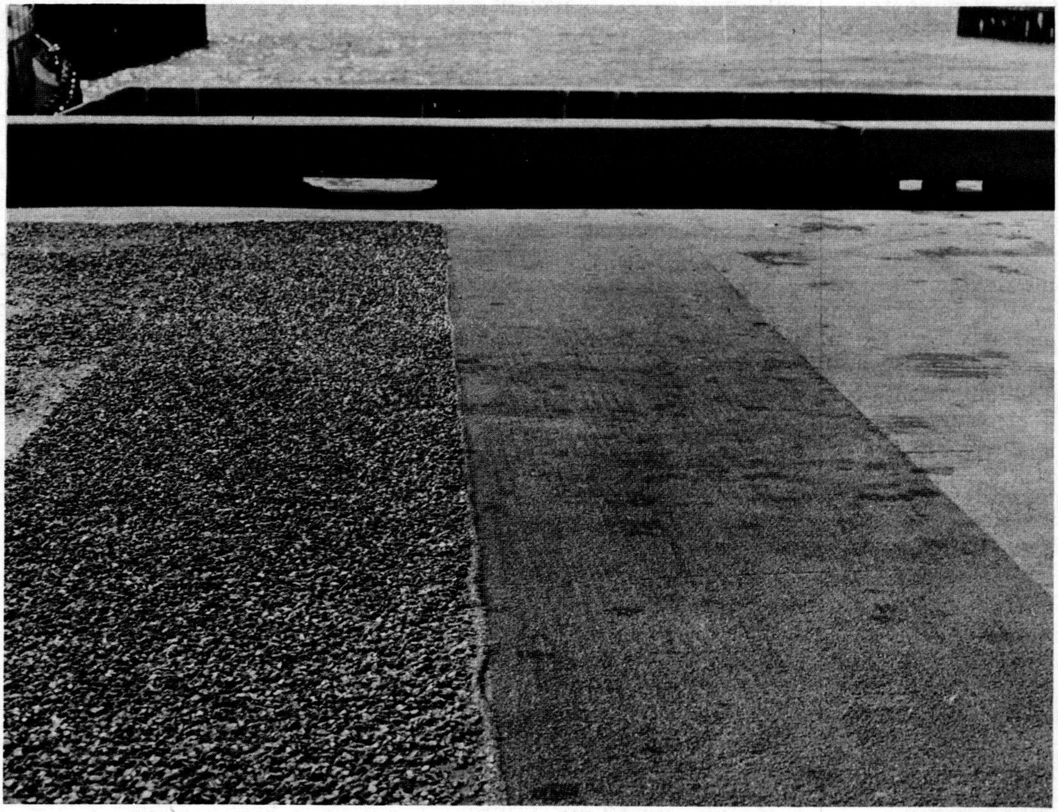

Figure 1. Condition after $2\frac{1}{2}$ yr service of large and small aggregate seal coats bonded directly to PCC with polyester resin.

selected for the particular result desired. Pneumatic-tired rolling is preferred because it gives good stone imbedment without grinding and breaking the chips. A sand "choke" follows, which provides a support to help keep the larger stones from whipping out if traffic is permitted before the resin is applied. Prior to the resin spray application, excess sand is removed by light mechanical sweeping. The sand choke, together with the chips, acts as a matrix which is bonded together by the resin. The sand further serves to absorb the resin before it reaches the underlying asphalt. This is necessary because polymerization of the resin is inhibited by contact with asphalt. A final sanding produces the desired surface texture. Multiple coats may be obtained by further application of resin and sand. Curing time can be adjusted readily from a few minutes to several hours, and thus no difficulty is encountered in opening the newly overlaid pavement to traffic. A final heavy sweeping is generally desirable to remove excess sand.

SELECTION OF MATERIALS AND APPLICATION RATES

The polyester resin and asphalt laminated systems can be easily adapted to give many different kinds of coatings. If an especially coarse-textured, high noise-level surface is needed to serve as an audible traffic warning, the initial layer is a 1-in. aggregate held in place by approximately 0.3 gal per sq yd of asphalt. Figure 5 is a close-up of a "rumble" surface. Unlike ordinary seal coats, the amount of asphalt used is not critical so long as it is sufficient to hold the stones in place until the resin is applied and is not so large that the stones are submerged in asphalt. Also, unlike ordinary seal coats, the type of asphalt used is, from limited data, not critical. Overlays have been service-tested using both cationic asphalt emulsion and hot 85/100 penetration paving asphalt. Both work well; however, the emulsion is preferred if it can be allowed to dehydrate.

If a thin, lightweight (¼-in.) overlay is required, the initial seal is made with ⅜-in. chips, or pea gravel, held by approximately 0.15 gal per sq yd of asphalt. Typical application rates, based on the Corporation's experience, are given in Table 1.

Because high durability is desired for these special overlays, care should be taken in aggregate selection. If antiskid properties are necessary, stones with high polishing resistance, or special antiskid materials, should be used. Clean, dry, "one-sized" aggregates are preferred. Grain size distributions of aggregates used in these experiments are given in Table 2.

Polyester resins were selected because of their great strength, chemical resistance, and excellent bonding qualities. These resins have low toxicity, cure well in the temperature range of 40-150 F, and have a viscosity convenient for spraying and mixing at ambient temperatures (3).

CONSTRUCTION EQUIPMENT

Procedures for making the initial seal coat with asphalt follow conventional practices and use readily available equipment. Asphalt emulsions or hot asphalt are spread by spraying with an ordinary distributor. Coarse chips may be spread by tailgating from

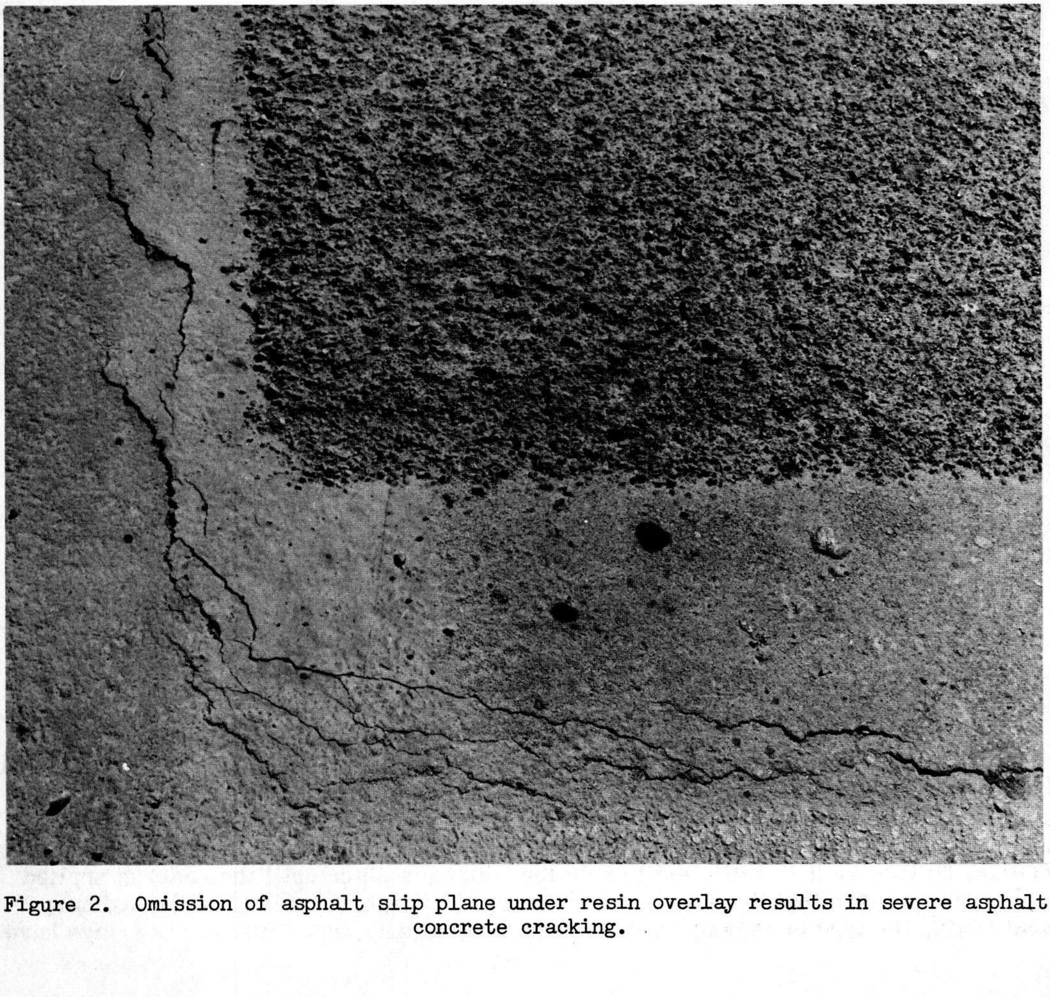

Figure 2. Omission of asphalt slip plane under resin overlay results in severe asphalt concrete cracking.

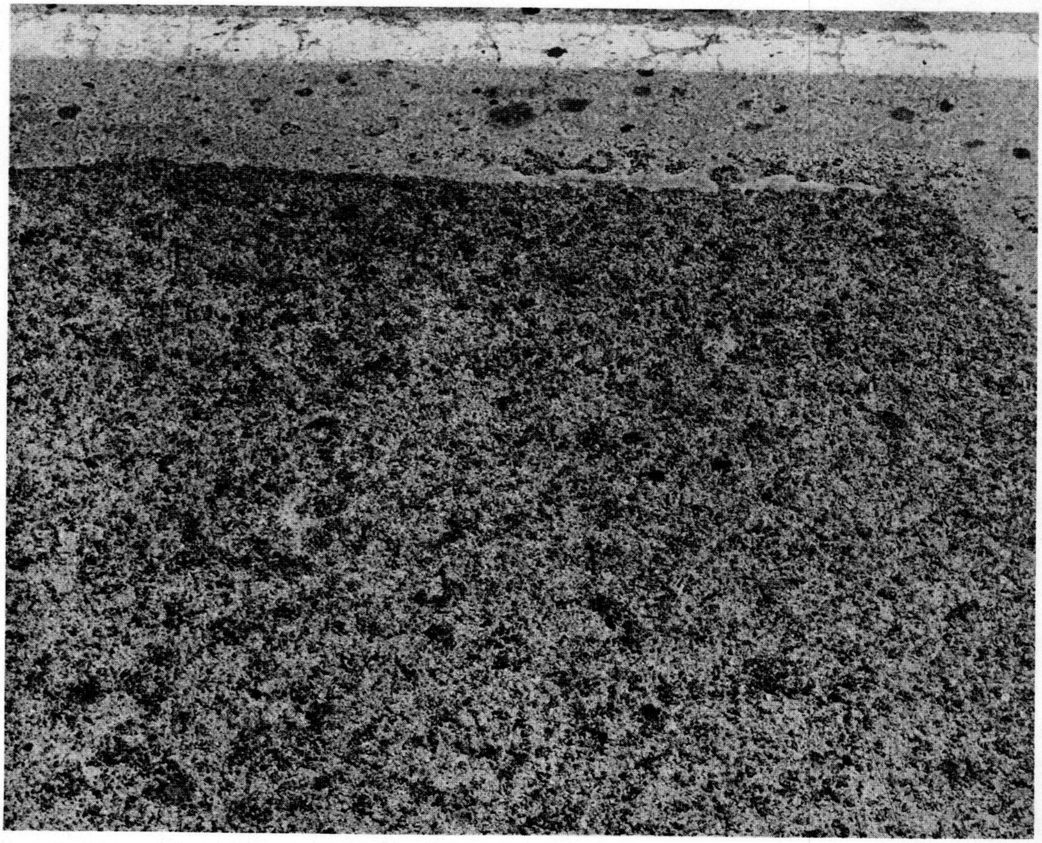

Figure 3. Properly used asphalt slip plane under polyester overlay. No sign of cracks in asphalt concrete after 9 mo. service.

a truck or, preferably, with a commercial self-propelled chip spreader. Sand application may be ty tailgating as well.

To obtain accurate and close control of aggregate spreading rates needed for experimental purposes, the small belt spreader shown in Figure 6 was constructed. It consists of a wheel-mounted, 10-ft wide hopper feeding a moving continuous belt. Belt and forward speeds are controlled by separate variable-speed electric drives.

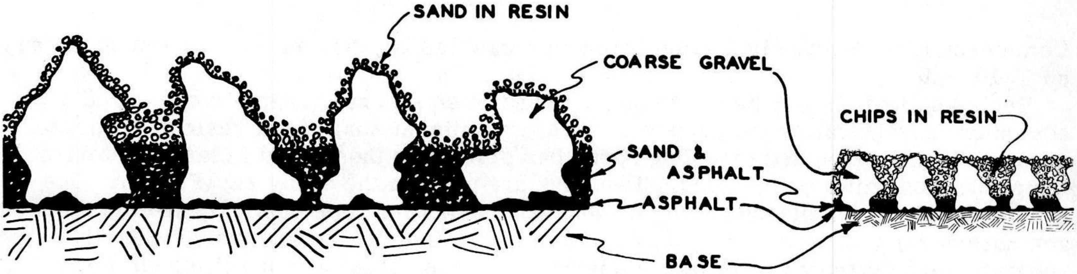

Figure 4. Sectional diagram of laminated overlays.

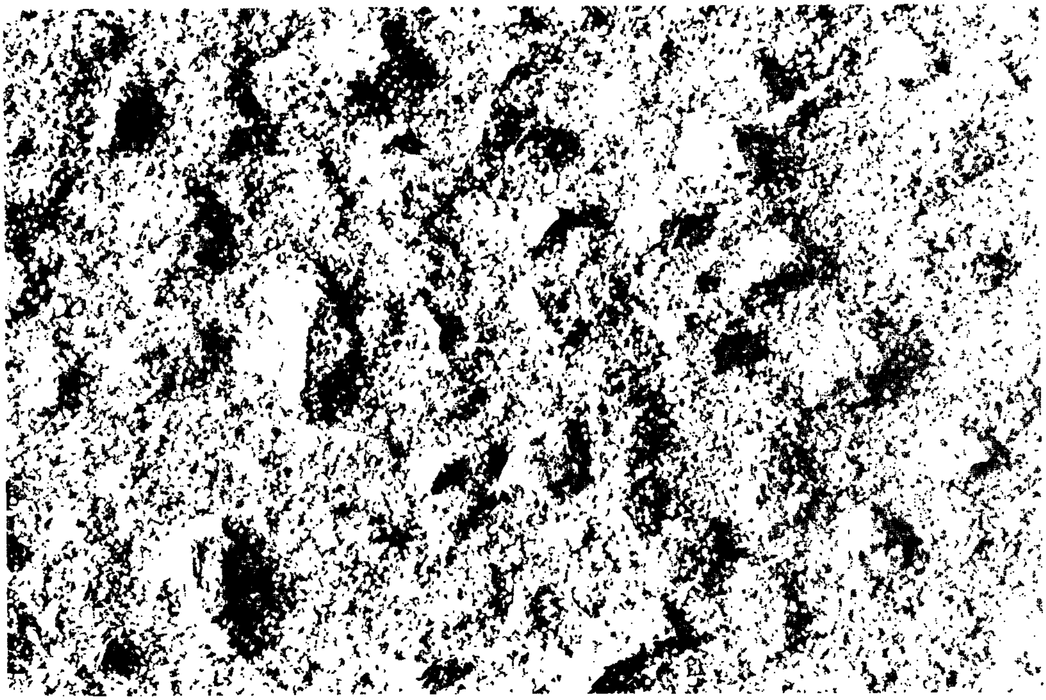

Figure 5. Rumble strip surface texture using 1-in. chips.

TABLE 1

MATERIAL REQUIREMENTS FOR POLYESTER RESIN AND ASPHALT LAMINATES

Overlay		Avg Asphalt Spray Rate	Coarse Aggregate		Polyester Resin Spray Rate	Fine Agg for Choke and Final Cover	
Types	Uses	(gal/sq yd)	Type	Rate (lb/sq yd)	(gal/sq yd)	Type	Rate (lb/sq yd)
Thin antiskid	Curves, intersections, slippery surfaces	0.15	3/8 x No. 4 chips	15	0.1	No. 8 grits	10
Thin overlay	Solvent resistance, prevent raveling	0.15	"Pea gravel"	15	0.1[a]	No. 8 sand	8
Armor coat	Heavy traffic areas	0.25	3/4- by 1/2-in. chips	20	0.2[a]	No. 8 grits or sand	10
Rumble strip	Safety warning	0.30	1- by 3/4-in. chips	30[b]	0.2	No. 8 sand	10

[a]Several courses of resin and fine aggregate may be applied.
[b]Use sparingly to obtain coarse texture.

Commercial, self-propelled chip spreaders can easily perform this function at speeds up to 10 mph.

Resin application may be performed by hand in small areas, using commercial equipment developed for the purpose. To apply uniform amounts of resin over large areas, a trailer-mounted traveling spray bar similar to those used in large factories for production painting was used. Two guns are used on the spray rig (Fig. 7). One gun sprays the resin blended with accelerator (for setting time control), and the other gun sprays resin blended with catalyst. In this fashion, the setting starts when the separate spray streams meet on the pavement. Thus, problems of setting up in the pumps, guns, or lines are minimized. The catalyst and resin mixture (without accelerator) is sufficiently stable for use within one or two days, and the accelerator and resin blend is stable for weeks. Ample time for equipment cleanout is available. The

TABLE 2

GRAIN SIZE DISTRIBUTION DATA FOR OVERLAY AGGREGATE

ASTM Sieve Designation	Square Opening (mm)	Cumulative Percent Passing by Weight					
		3/8-in. Chips	"Pea Gravel"	3/4-in. Chips	1-in. Chips	No. 8 Grits	No. 8 Sand
1 1/4	31.7	-	-	-	-	-	-
1 in.	25.4	-	-	-	100	-	-
3/4 in.	19.1	-	-	100	92	-	-
1/2 in.	12.7	-	-	37	31	-	-
3/8 in.	9.52	100	100	24	5.6	-	-
No. 4	4.760	56	91	3.0	0.2	-	-
No. 6	3.327	28	1.0	1.3	-	100	-
No. 8	2.380	8.0	0	0	-	81	100
No. 10	2.000	4.0	-	-	-	39	93
No. 16	1.190	0	-	-	-	0	24
No. 20	0.840	-	-	-	-	-	1.0
No. 30	0.590	-	-	-	-	-	-

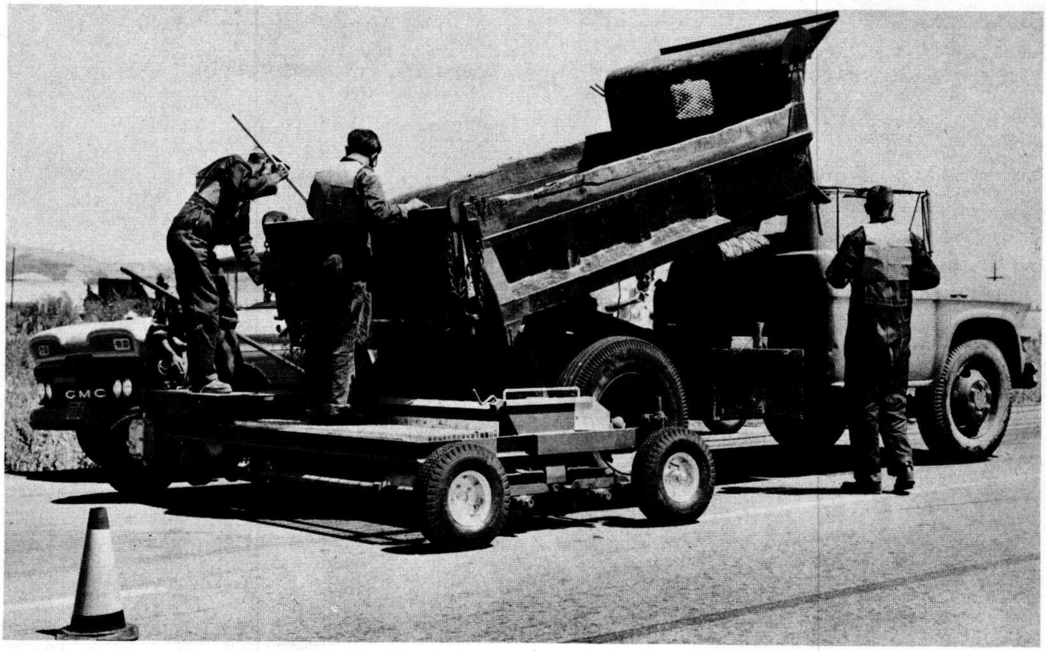

Figure 6. Loading chip spreader with aggregate.

separate guns are supplied with resin under pressure from two air-operated pumps. Pumps and resin containers are placed in the spray rig (Fig. 8). Application rates are controlled by pump pressure and forward speed of the rig.

To obtain the close control of spray rates desired for their experiments, forward movement of the spray rig is adjustable by a variable-speed electric drive from 0-20 ft a minute. Transverse speed of the moving guns is also adjustable, but 240 ft a minute has proved to be a practical rate. Automatic gun shutoff is easily adjusted to control spray widths from 2-10 ft. Application rates are independent of the spray width because the spray gun movement is always the full width of the spray bar.

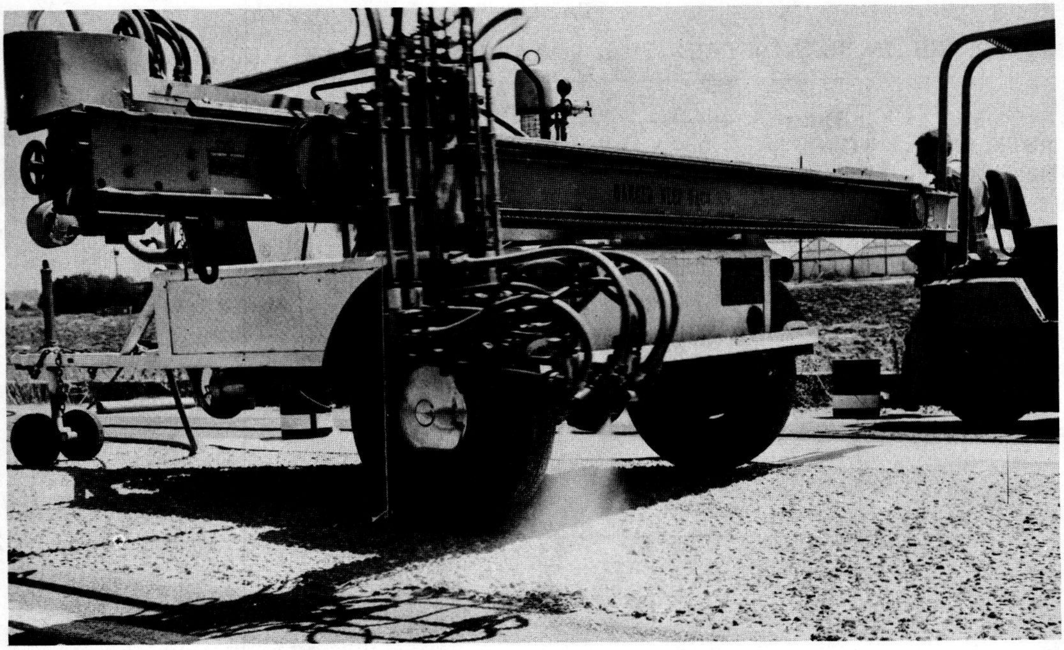

Figure 7. Resin spray rig in operation on rumble strip.

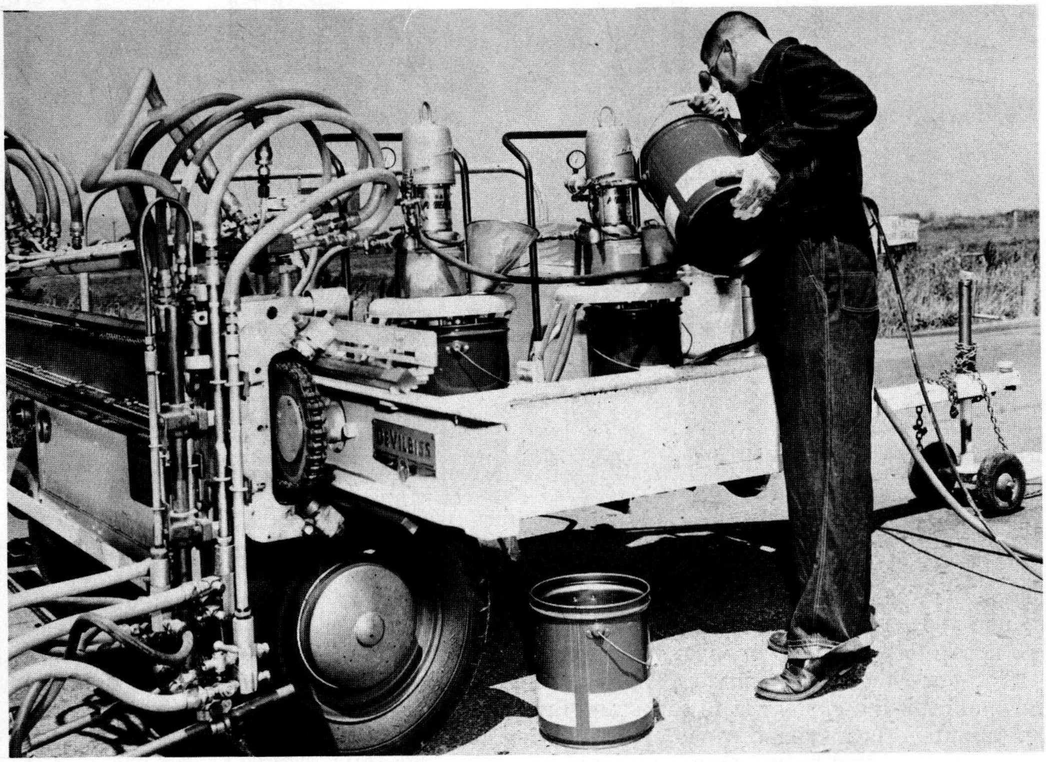

Figure 8. Filling spray rig resin containers.

This method, using traveling spray guns, is preferred to a gang of multiple spray heads because of ease of calibration and control for experimental work. Where very large areas are to be coated rapidly, twin spray bars with multiple heads may be used.

Mixing the resin with catalyst and accelerator was done at the work site just prior to spraying. For large applications, automatic proportioning and blending equipment may be used. Recently, spray guns have been developed which meter catalyst into the resin while spraying. These should be especially useful on large projects, because premixing resin and catalyst is eliminated. Experimental procedure has been to carry a barrel of resin and a laboratory mixer in the back of a pickup truck preceding the spray rig down the road. This truck also tows an air compressor and a gasoline-driven electric generator (Fig. 9).

RUMBLE STRIP CONSTRUCTION

The first public demonstration of the durability of this new system was made on a public highway in Contra Costa County, California. The purpose was to install a traffic safety improvement called "rumble strips," designed by the County Public Works Department. The basic objective is to alert drivers, thus slowing down fast-moving cars approaching dangerous intersections or other hazards. This is accomplished by placing 25-ft long pads of rough-textured aggregate at 50- to 100-ft intervals on the appropriate lane. Previous experience with conventional rumble strips in Cook County, Illinois (4), as well as elsewhere in Contra Costa County, showed that asphalt binders do not retain sparsely placed, large-size aggregate for prolonged periods under heavy traffic unless it is fully choked with smaller-size aggregate or is a part of a multiple seal coat. This is an especially serious problem because the initial hazard may be increased rather than diminished by loose stones and slippery surfaces.

This first installation was made during July 1960 on Third Street near Paar Boulevard outside of Richmond, California. A major portion of the finished job is shown in Figure 10. The materials and quantities used are given in Table 1, except that the asphalt rate was 0.4 gallon per square yard. Aggregate gradings are shown in Table 2. Six pads were constructed on AC, two pads on the PCC bridge deck, and an antiskid pad at the

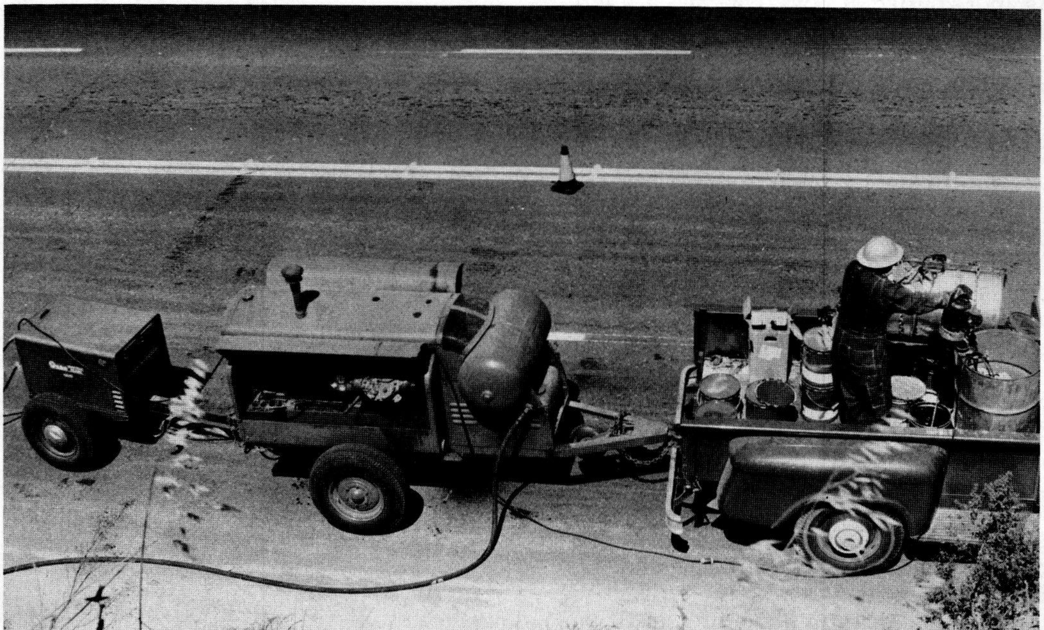

Figure 9. Truck for resin blending tows air compressor and generator.

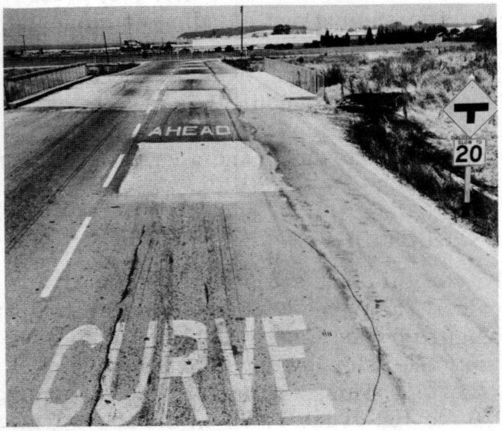

Figure 10. Several completed rumble strips north of Richmond, California.

intersection using the same general procedure. The materials recommended in Table 1 for antiskid pads were used. The asphalt rate was 0.25 gal per sq yd, and the resin rate was 0.20 gal per sq yd.

A second group of rumble strips (Fig. 11) was opened to traffic in August 1960 in the northbound lanes of San Pablo Avenue (business, US 40), approaching a stop sign south of Rodeo, California. In addition, an antiskid pad was installed on the last 90 ft before the stop sign. The 12 strips were designed as an experiment to study the following variables: type and amount of polyester resins, amount of asphalt, size and shape of aggregate, and the use of fine fillers in the resin. Some sections were purposely underdesigned to determine optimum conditions.

The several kinds of polyester resins were included in this field test to establish the optimum resin properties for the overlay system. The characteristics of the resins used are given in Table 3. Resins A and B are high impact types and C and D, rigid types. In the Rodeo field test, all four resins were applied at rates from 0.15 to 0.3 gal per sq yd. Resin A was applied at rates as high as 0.5 gal per sq yd. In some of the experiments these resins were extended with as much as 60 percent limestone dust filler (-325 mesh).

Most of the large chips used were the 1- by ¾-in. size described in Table 2. However, some experiments included a ¾- by ½-in. crushed granite or a 1- by ¾-in. uncrushed gravel. In these particular experimental sections the initial asphalt bond between the AC base and the large chips was made using hot 85-100 penetration grade asphalt applied at both 0.25 and 0.4 gal per sq yd. Also included for control purposes in this series of experiments was a section made without any resin.

PERFORMANCE OF RUMBLE STRIPS

The Richmond rumble strips are subjected to a traffic count of 900 vehicles a day (5). With the minor exceptions noted in the following paragraph, all of the strips are in excellent condition after five months' service and show no loss of stones.

There is a slight indication of asphalt bleeding through the porous resin-sand matrix on the PCC bridge deck. Apparently less asphalt may be used to bond chips to PCC. There is no evidence of this in the sections on the AC. There is no evidence of AC cracking adjacent to the coatings except in a few places where the resin was accidentally applied directly to the AC. The slip plane of asphalt between the base and the overlay appears to prevent cracking. Two cracks appeared

Figure 11. Rumble strip experiments south of Rodeo, California

TABLE 3

PROPERTIES OF POLYESTER RESINS USED
(Clear Casting Types Containing Styrene)

Properties	Resin A	Resin B	Resin C	Resin D
Barcol hardness	33-36	17	40-45	30-35
Flexural strength, psi	18,000	11,800	15-20,000	13-17,000
Flexural modulus, psi	4.8×10^5	2.9×10^5	5.4×10^5	6.2×10^5
Impact strength, ft-lb/in.	6-8	3-4	2.5-3	2.6
Heat distortion temperature, deg F	174	127	240-250	198
% styrene in resin	40	40	40	33

across the center of the antiskid sections. These appeared before the resin had completely cured and before traffic was allowed on the sections. This cracking appears to have occurred because an excess of resin was used which resulted in an abnormal amount of shrinkage during curing.

High noise level rumbles are obtained with the 1- by ¾-in. granite chips. At 50 mph on the uncoated pavement in a car with windows closed, the background noise level by the driver's ear was 94 decibels. As the car crossed each strip, the noise level increased to an average of 104 decibels for a fraction of a second. The significant increase in noise level, together with slight vibrations through the steering wheel, is repeated each time the driver crosses a rumble strip before the intersection. A heavily laden truck was observed to lock its wheels over this section to avoid an accident. Except for streaks of rubber, no damage was evident.

The Rodeo rumble strips are subjected to approximately 1,000 vehicle passages per day (5). After five months' traffic, periodic inspections of the nearly 200 separate tests resulted in the following conclusions:

1. In the few sections where no resin was applied, stones were lost from both the high- and low-rate asphalt application areas. Many of the stones were lost in the first two weeks. In all of the resin-coated test strips, no loss of stones was evident.
2. No cracking of the AC base is evident around the edge of the pads.
3. No cracking in the overlay was evident where moderate resin application rates were used. However, cracks appeared where large and excessive amounts of resin were used, that is, greater than 0.3 gal per sq yd.
4. Where these large amounts of resin were used, cracking was more extensive when filler content was high, presumably because the higher viscosity of the filled resin prevented adequate penetration into the sand matrix.
5. No cracks were seen in filled resin strips where spray rates were moderate, that is, 0.3 gal per sq yd or less.
6. Where sand and resin application rates were highest, rumble was low because of the relatively smooth surface.
7. At this time there do not appear to be any differences in the behavior of the four resins used. This comparison includes all resins both unfilled and filled at equal levels. At a later time, significant differences may appear.

Ratings of these test sections will be continued for several years. Present application recommendations (Table 1) emphasize durability of the overlay. However, as shown by preliminary results, substantially lower resin quantities, as well as the use of extenders, may be permissible without appreciable sacrifice in durability.

OTHER USES FOR RESIN AND ASPHALT LAMINATED OVERLAYS

As mentioned in earlier paragraphs, this new system is easily adapted to various purposes. For example, although no precise measurements of skid resistance have been made at the present installations, it is evident that there is a substantial increase in friction of the antiskid pad compared to the original pavement. Retention of high antiskid properties depends on proper aggregate selection (6). In addition, the sawtooth nature of the surface provides deep channels for water drainage and should reduce the amount of wiping action required of tire treads for high friction in wet weather.

Suitable coatings can be provided where thin, low-weight overlays are required because of dead weight or grade change limitations. Thus, commercial floors, bridge decks, and wharf decks may be benefited by the proper laminated overlay.

Although not net field tested, preliminary work shows that coatings highly impervious to solvents can also be made. Using a chemically inert resin, no damage either to the overlay or to the surface below will occur as the result of fuel spills or special washing procedures. The resin is thermosetting, and no damage by jet blasts is expected because of the exceptional strength of the system and the heat resistance of the resin.

The high tensile strength of these overlays may be of value in the structural design of flexible pavements. By bonding a resin system to both the upper and lower surfaces of an AC layer, a composite action may be obtained which greatly increases the modulus of elasticity of the surface layer. The principle would be similar to the action occurring in a wooden beam having steel flanges. The exceptional strength of laminated aircraft wing structures also depends on the same principle. Preliminary tests show this concept to be sound.

Application to emergency or military problems in stabilizing loose sands should not be overlooked. Polyester resin viscosities are low, and positive setting is rapidly attained at ambient temperatures above 40 F. Therefore, simple penetration treatments are possible.

CONCLUSIONS

Laminated resin and asphalt overlays have demonstrated their use as durable coatings under heavy vehicles and where special surface textures are required. Either large or small particles can be held firmly in place against intense traffic abuse. Rumble strip durability shows significant improvement over past practice using asphalt alone.

Conventional construction procedures and equipment are used for the most part to construct the new system; therefore, special training is not required. Although a spray rig is needed, the components are all commercially available. In addition, simpler methods for resin application are available if there is not the need for the experimental flexibility designed into the equipment as described in this paper.

All of the equipment used is portable, independent of large central plants, and may be used in remote areas. Because of this and the absence of need for pretreatment of existing surfaces, installation costs are minimum. While the initial cost of resins is high compared with those of cement and asphalt, resins have desirable properties that neither of these other materials possess. Thus, annual cost of the laminated overlay system may prove substantially lower than that of other alternatives. A number of additional uses for the resin and asphalt-laminated overlay are suggested as a result of the present installations in California.

ACKNOWLEDGMENTS

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