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Number 236

Surface Treatments, Skid Resistance and Aggregates

6 Reports

	Subject Area
26	Pavement Performance
31	Bituminous Materials and Mixes
35	Mineral Aggregates
40	Maintenance, General

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Foreword

These papers deal with applied research on bituminous seal coats and hot plant mix. The information will be of interest to highway paving engineers and also to those doing research in these areas.

In his attempt to improve the performance of seal coats in South Dakota, Crawford compared and evaluated the use of rapid setting asphalt, cutback asphalt and a rubberized asphalt. He concluded that while there was some difference in performance, the construction control, weather, and equipment were far more important than the type of asphalt used.

The problems of construction control with seal coats were the subject of a series of study undertaken by Mahone and Runkle. They concluded that the distribution of the asphalt can be checked by a cup test for a study of nozzle output, a trough test for a study transverse distribution, and a cotton pad test for either transverse or lateral distribution in the field.

Skid resistance is the subject of two studies. Zube and Skog conclude that a satisfactory correlation between a laboratory wear and polish apparatus and wear and polish under traffic has been established. The apparatus consists of a regular tire that is rotated at 13 rpm on a stationary test panel. Burnett, Gibson and Kearney conclude that only after 5 million vehicle trips over a section does the coefficient of friction remain stable, and decisions about the mix should not be made prior to that time. They also conclude that it is the coarse aggregate rather than the fines that control the skid resistance of bituminous mixes.

Gallaway and Harper concluded from a laboratory study that lightweight aggregates can be blended with conventional aggregates to provide good quality bituminous hot mixes. Laboratory compaction with the gyratory shear apparatus degradation was found to be insignificant. The laboratory results suggested that asphalt absorption of 2 to 3 percent is likely in the field.

The paper by Tons and Goetz deals with the weaknesses of the conventional sieve gradations serving as a vital parameter of bulk properties. They conclude that the volume which a piece of aggregate occupies in a mass of particles is a more meaningful parameter of bulk properties. They suggest that the packing volume can be defined by particle geometry and surface area and rugosity.

—Jack H. Dillard

Contents

South Dakota Chip Seal Coat Study

ROBERT A. CRAWFORD, South Dakota Department of Highways

This study is concerned with the comparison and evaluation of chip seal coats placed using an RS-2 emulsified asphalt, an RC-4 cutback asphalt and several rubberized RC-4 asphalts. All comparisons and the evaluation are based on ratings determined by the "South Dakota Seal Coat Evaluation Procedure" (2). The RS-2 chip seal coats did not perform as well as the RC-4 and rubberized projects. The addition of rubber to asphalt of the type used in South Dakota from 1956 to 1959 does not improve either the quality or life expectancy of chip seals. For this reason rubberized asphalts are not considered to be economically feasible at the present time. Other observations concerning chip seal performance were made during the study. It is concluded that construction control, weather, equipment and the experience of construction personnel are variables which apparently have more profound effects on a seal coat than the type of asphalt used.

•CHIP seal coats in South Dakota have primarily been accepted as a means of protecting the surface of bituminous mats from oxidation or polymerization of the bituminous cement exposed at the surface. As the name implies, it was also felt that chip seals sealed the surface to prevent the entrance of moisture into the various component layers of the pavement from the top. At the same time a surface could be produced which apparently resulted in better frictional relationships between vehicle tires and road surface and also produced better night visibility characteristics of the surface. Other benefits were attributed to seal coats by some individuals, such as enlivening an old or dry mat and strengthening a pavement to a limited extent as a result of a small increase in thickness.

Over the years a more or less standard construction procedure was developed for chip seal coats. The standard seal utilized RC-4 asphalt cutback with either ½ or ¾-in. maximum size crushed rock or crushed gravel chips. The basic construction procedure consisted of spraying sufficient RC-4, with 1.0 percent no-strip additive, on the mat surface to cause the chips to stick but not so much as to cover them. It was assumed that the RC-4 would stick to the mat and that the chips would stick to the RC-4. In a number of cases these assumptions were found to be faulty. At times, chips would be whipped off the surface by traffic causing damage to vehicles and nullifying many benefits of the treatment.

In 1955, a decision was made to try a cutback asphalt which had rubber added to it. The rubber, it was thought, would much improve the adhesive qualities of the asphalt and thus prevent aggregate loss, among other things. As a result of the good showing of the one 1955 rubberized chip seal project a comprehensive program of sealing using rubberized asphalt was initiated in 1956.

The State did not specify the type of rubber nor the percentage to be incorporated in the asphalt on many of the projects. It is believed that most projects utilized 3 percent styrene-butadiene rubber incorporated in RC-4 cutback asphalt but no records are available in Highway Department files to substantiate this belief.

Paper sponsored by Committee on Bituminous Surface Treatments and presented at the 47th Annual Meeting.

One project was placed in 1958 with approximately 10, 000 gallons of each of the following asphalts: RC-4 with 1. 0 percent no-strip additive which was the standard bitumen, RC-4D with 3. 0 percent styrene-butadiene, RC-4DN with 1. 5 percent neoprene and RC-4DNR with 1.5 percent natural rubber (1). This job was placed to determine the relative merits of the various types of rubber as compared to the regular RC-4.

From 1956 to 1959, inclusive, over 800 miles of rubberized asphalt chip seals were placed. During this same period, about 400 miles of chip seals were placed without utilizing the rubber additives. The cost of the rubberized seal coats averaged about $375 per mile more than the nonrubberized projects. In addition to these sections, it was decided in 1959 to place about 200 miles of chip seals using RS-2 emulsified asphalt without rubber. The emulsion increased the cost by approximately $600 per mile over the regular seal using plain RC-4.

To determine the relative merits of these different asphalt materials for chip sealing as well as to evaluate chip seal coats in general, a method was devised for determining the condition of seal coats numerically (2). Over the next six years, the 1400 miles of chip seals were rated periodically to determine which chip seals had the higher condition rating and which chip seals had a longer life.

EVALUATION SYSTEM

In the early planning stages of this study a search was made for a method of evaluating chip seals. Several methods existed but they entailed taking photographs of small areas of the surface or sweeping up chips from the shoulder, or they produced ratings of many properties individually. With 127 projects to rate consisting of over 1400 miles of road scattered over the entire State it was obvious that a fast method of rating was needed. The need to compare these projects required easily tabulated, reproducible and comparable data from each project. None of the existing systems of rating chip seals could meet these demands. As a result a new rating system was developed which has provided the data for this study. This procedure has been published elsewhere and will not be described in detail in this report (2).

In general the system involves only visual inspection and five factors are rated independently: (a) chip retention, (b) skid resistance, (c) uniformity of application, (d) cracking, and (e) bleeding. Through the evaluation of these factors it is not only possible to determine how well a seal coat is performing its major functions but also to determine in which of these five categories various types of seal coats excel.

In rating a seal coat, it has been found that better agreement between raters was obtained if small sections of highway were rated separately and an average taken as the rating for the project. The length of section can be varied depending on the length of the project and the time available for rating. A length of one mile was selected as a practical rating length in South Dakota for ordinary projects.

Each of the five factors is rated independently from 0 to 20. A perfect chip seal will then receive a total value of 100 if it rates 20 in each of the five categories being rated. During the study four men were used to rate the 127 projects involved. Of the 127 projects scheduled for rating only 120 are included in this report. Projects No. 29, 43, 68, 83 and 95 were not sealed as scheduled and could not be included. Project 119 is the only 1955 rubberized job and is not included. Project 121 is the test section using RC-4 plain with three different types of rubber added; it is discussed separately.

Two men independently rated each section each year. An average of both ratings was taken as the project rating. On new projects agreement between the two raters was generally excellent. In some cases of serious disagreement, both raters revisited the project in question and worked out the differences together. These differences probably were the result of inattention or fatigue on the part of one rater on new projects. However, as the projects got older some serious disagreements on basic concepts resulted.

Chip retention presented a problem when bleeding was severe. One school of thought held that you could not have serious bleeding and loss of chips together because if there was an excess of asphalt the chips would be retained even though they could not be seen. The other held that if no chips were visible on the surface of the road they were contributing no value to the chip seal and for all practical purposes could be considered as missing. At least one example was found where the asphalt stuck to the old mat but the chips did not stick to the asphalt. In this case the surface was very rich and could be said to have bled and at the same time chip retention was very poor or nonexistent. It was finally decided that the question of whether the chips had been retained in a bleeding chip seal was not the real issue. The main point was whether the chips were serving their intended function. In cases of serious bleeding they definitely were not and a reduction in the rating value assigned to chip retention was in order.

Skid resistance did not present a problem in itself; however, it was argued that if a road was slippery because of bleeding the rating for bleeding was lowered, the rating for skid resistance was lowered, and the rating for chip retention was lowered—resulting in excessive penalizing of a bleeding job. In South Dakota, chip seals are frequently placed for improvement of skid resistance and for better night visibility. For this reason, it was felt that a bleeding chip seal was nearly a complete failure and could not be overpenalized. At least one project seemed to improve after having serious bleeding initially. The excess asphalt apparently was removed by oxidation or other means over several years, and the chips finally began showing on the surface and after eight years the project looked fairly good.

Uniformity of application caused much discussion on older projects. One opinion held that uniformity of application could be called uniformity of appearance and if the surface did not appear to be uniform in appearance it should be rated down. Another held that cracks which had been filled in the old mat or rich patches placed on the old mat would bleed through the chip seal and make the surface appear rich in places through no fault of the chip seal. It was also felt that a patch over the seal coat would make the surface nonuniform in appearance although the application may have been very uniform originally. It was agreed that any nonuniformity which could not be traced to the original application should not influence the rating.

There has been much discussion as to whether cracking should be rated at all. Although the name chip seal implies that the treatment will seal the surface against the entrance of moisture it has been found that this benefit is nonexistent after a few months. Any crack in the old mat will reappear the first spring following application. Cracking, especially alligator-type cracking, is due to deep-seated weaknesses which can never be corrected by a thin overlay of chip seal. In other words, cracking is caused by deficiencies in the road which cannot be corrected by a chip seal. This finding seemed to be contrary to popular opinion, so cracking was left in the rating system in order that skeptics could convince themselves that a seal coat does not seal cracks. On some older projects, areas of severe cracking had been corrected by patching. For this reason, patched areas were assumed to have been cracked areas. Therefore, a heavily patched road was rated down for cracking although maintenance had corrected or temporarily alleviated the condition.

On some projects particular care had to be taken in assessing bleeding. Blackening of the wheel lanes is at times due to bleeding, but on other projects it may be due to tire wear or streaking from traffic. Disagreements as to bleeding could usually be reconciled through a close look at the surface. Bleeding problems as related to other rating factors have been discussed earlier and will not be repeated here.

It was apparent that even with these differences of opinion a standardized approach could be arrived at and the rating system seemed to provide reliable and reproducible results.

DISCUSSION OF DATA

As stated previously, the annual rating given each project is an average of two ratings by two individuals who evaluated each project independently. The raw data obtained by each individual rater during this study are available from the Highway Research Board.[1]

[1]These data are available by special arrangement for cost of reproduction and handling. Inquiries should refer to XS-17, Highway Research Record 236.

TABLE 1

AVERAGE RATINGS OF PROJECTS CONSTRUCTED IN 1956

Proj. No.	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
(a) RC-4						
86	86	84	75	68	77	68
114	87	84	82	74	78	74
124	76	60	60	52	50	54
(b) RC-4D (Rubberized)						
3	96	87	85	68	72	70
7	74	58	56	54	59	—[b]
24	80	59	64	54	59	50
34	68	56	53	—[b]	54	55
37	66	51	52	—[b]	61	59
42	74	74	61	56	56	58
45	78	70	72	64	—[b]	—[b]
46	86	78	77	76	72	72
48	78	73	60	51	—[b]	—[b]
50	74	72	65	62	68	59
51	86	82	77	72	72	67
92	84	84	70	64	72	66

[a]In 1963 the rating system was revised mainly in the method of evaluating cracking; this revision resulted in a general increase in the 1963 ratings over those obtained in 1962.

[b]Evaluation discontinued when rating dropped below 50.

TABLE 2

AVERAGE RATINGS OF PROJECTS CONSTRUCTED IN 1957

Proj. No.	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
(a) RC-4						
58	67	70	57	53	—[b]	—[b]
59	94	85	74	73	72	70
63	91	87	82	76	78	72
69	94	88	78	76	80	75
93	94	86	77	72	68	70
100	86	82	71	62	74	73
113	84	74	70	63	66	62
127	73	57	50	—[b]	—[b]	—[b]
(b) RC-4D (Rubberized)						
20	68	58	50	—[b]	—[b]	—[b]
21	84	54	56	56	57	50
22	70	62	55	—[b]	59	—[b]
30	83	67	68	54	60	50
62	82	72	68	—[b]	—[b]	—[b]
64	96	86	85	74	69	64
70	78	77	72	64	64	61
74	57	58	—[b]	—[b]	—[b]	—[b]
111	75	56	59	—[b]	—[b]	—[b]
116	82	74	60	54	—[b]	—[b]

[a]See footnote a, Table 1.
[b]See footnote b, Table 1.

TABLE 3

AVERAGE RATINGS OF PROJECTS CONSTRUCTED IN 1958

Proj. No.	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
			(a) RC-4			
2	93	76	73	72	72	72
15	77	64	52	59	52	50
26	72	64	56	—[b]	54	—[b]
28	82	71	72	68	72	66
33	79	78	64	52	—[b]	—[b]
36	73	77	74	60	57	—[b]
54	94	86	82	74	73	73
55	86	62	52	—[b]	56	57
56	98	86	88	77	75	66
65	93	89	80	72	74	68
67	63	—[b]	—[b]	—[b]	—[b]	—[b]
73	81	57	64	—[b]	58	55
87	80	74	70	59	60	64
102	90	86	84	72	78	63
103	53	—[b]	—[b]	—[b]	—[b]	—[b]
105	92	90	77	72	70	72
107	81	76	68	58	62	60
109	88	83	76	68	73	71
115	94	84	82	71	76	71
117	95	81	80	70	71	70
123	74	72	64	55	—[b]	—[b]
			(b) RC-4D (Rubberized)			
4	89	80	70	56	58	—[b]
9	73	58	50	—[b]	—[b]	—[b]
11	66	54	58	—[b]	—[b]	—[b]
19	73	—[b]	—[b]	—[b]	—[b]	—[b]
23	74	—[b]	—[b]	—[b]	—[b]	—[b]
25	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]
49	53	—[b]	—[b]	—[b]	—[b]	—[b]
72	86	75	80	64	65	64
81	79	67	54	58	58	54
94	71	52	54	—[b]	—[b]	—[b]
98	68	—[b]	—[b]	—[b]	—[b]	—[b]
120	78	76	68	58	66	66
122	70	62	62	51	57	51

[a]See footnote a, Table 1.
[b]See footnote b, Table 1.

Table 1 lists the average ratings of those projects constructed in 1956. Ratings were made in 1959, 1960, 1961, 1962, 1963 and 1964. Table 2 presents the same data for projects constructed in 1957; Table 3, for those in 1958; and Table 4, for 1959. When the average rating of a project dropped below 50 no further ratings were made. A rating of 50 indicates that the seal is no longer performing the tasks for which it was constructed and can be considered to have failed.

No ratings were made during 1965 and 1966. During 1967 an attempt was made to evaluate the projects; however, after rating about 50 of them the study was abandoned. By this time most of the projects were extensively patched and the ratings were influenced more by the condition of the entire highway structure than the chip seal alone. It had become obvious by this time that a chip seal that was going to fail had generally deteriorated to a large extent by the end of the first year. If at the end of the first year the seal rated relatively high it would continue to be a good seal for years to come provided it had been placed on a structurally sound pavement. In other words, there is no point in rating a seal before it is about one year old and there is not much reason to evaluate a seal after it is older than one year. After the seal is one year old it has withstood the initial test of traffic and climate, has not yet required extensive patching or other maintenance activity and yet has allowed cracking to develop. This is considered to be the best time to conduct a condition survey. A seal rated immediately after construction has not had sufficient traffic or climatic influence to test its adequacy.

TABLE 4

AVERAGE RATINGS OF PROJECTS CONSTRUCTED IN 1959

Proj. No.	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
(a) RC-4						
32	96	82	79	66	77	74
40	90	72	68	52	58	58
41	94	75	71	59	68	69
60	94	72	72	58	51	—[b]
75	93	70	62	65	68	64
77	100	86	81	64	72	68
79	83	—[b]	—[b]	—[b]	—[b]	—[b]
80	87	52	55	54	60	56
104	90	88	82	74	78	69
106	98	92	86	74	76	72
108	89	86	78	75	79	76
112	98	94	92	80	83	78
(b) RC-4D (Rubberized)						
14	78	70	58	56	52	56
16	98	87	83	80	72	72
17	80	73	67	60	68	68
18	96	84	80	76	72	74
27	80	65	61	51	58	—[b]
31	96	80	80	71	75	80
35	98	86	80	56	68	66
38	94	76	74	64	62	56
39	96	60	60	58	59	59
44	98	86	76	70	76	76
47	88	54	—[b]	—[b]	52	—[b]
53	88	79	64	60	70	60
61	88	70	57	—[b]	58	—[b]
76	99	67	67	63	59	60
78	98	84	74	69	73	70
82	97	85	74	66	77	72
84	92	80	74	62	64	66
85	93	60	60	50	52	51
88	95	64	66	59	62	62
96	70	68	54	—[b]	54	—[b]
101	94	88	74	69	72	63
118	98	82	83	81	80	80
125	92	76	68	56	60	—[b]
126	84	72	65	62	69	63
(c) RS-2						
1	96	67	58	—[b]	—[b]	—[b]
5	99	86	86	72	78	74
6	72	62	54	—[b]	—[b]	—[b]
8	88	70	62	52	57	54
10	88	66	66	56	50	50
12	84	64	55	—[b]	—[b]	—[b]
13	94	78	73	63	66	54
52	88	68	52	—[b]	54	52
57	88	64	53	—[b]	—[b]	—[b]
66	82	72	70	55	—[b]	—[b]
89	93	72	64	52	58	58
90	96	69	69	56	53	54
91	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]
97	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]
99	86	60	50	—[b]	—[b]	—[b]
110	80	70	70	63	60	57

[a]See footnote a, Table 1.
[b]See footnote b, Table 1.

TABLE 5

CHIP SEALS CONSTRUCTED IN 1956

Item	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
Age	3	4	5	6	7	8
(a) RC-4						
Avg. rating of projects	83	76	72	65	68	65
Remaining in service:						
No. of projects rated	3	3	3	3	3	3
No. of projects failed	0	0	0	0	0	0
% of projects failed	0	0	0	0	0	0
(b) RC-4D						
Avg. rating of projects	79	70	66	62	64	62
Remaining in service:						
No. of projects rated	12	12	12	10	10	9
No. of projects failed	0	0	0	2[b]	2	3
% of projects failed	0	0	0	17	17	25

[a]In 1963 the rating system was revised mainly in the method of evaluating cracking; this revision resulted in a general increase in the 1963 ratings over those obtained in 1962.

[b]The two projects which rated below 50 during 1962 both rated above 50 in 1963 under revised rating system.

On the other hand ratings after two or more years are very likely to be influenced by maintenance activity. Patches are frequently placed over areas of poor chip seal and that portion of the seal remaining would rate higher than it should.

Tables 5 through 8 give the average ratings of all chip seals constructed with a given type of asphalt during a single year. Table 9 gives the same information for all seals constructed in all years. The number of each type of project rated and the number and percent of projects which have failed are also given. Table 10 lists the projects giving their location, length and other information.

The project using regular RC-4, styrene-butadiene, neoprene and natural rubber has been observed annually since August 1958 when it was constructed. There were

TABLE 6

CHIP SEALS CONSTRUCTED IN 1957

Item	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
Age	2	3	4	5	6	7
(a) RC-4						
Avg. rating of projects	85	79	70	68	73	70
Remaining in service:						
No. of projects rated	8	8	8	7	6	6
No. of projects failed	0	0	0	1	2	2
% of projects failed	0	0	0	12	25	25
(b) RC-4D						
Avg. rating of projects	78	66	64	60	62	56
Remaining in service:						
No. of projects rated	10	10	9	5	5	4
No. of projects failed	0	0	1	5[b]	5	6
% of projects failed	0	0	10	50	50	60

[a]See footnote a, Table 5.

[b]One of the projects which rated below 50 in 1962 rated above 50 in 1963 under revised rating system.

TABLE 7

CHIP SEALS CONSTRUCTED IN 1958

Item	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
Age	1	2	3	4	5	6
(a) RC-4						
Avg. rating of projects	83	77	71	66	67	65
Remaining in service:						
No. of projects rated	21	19	19	16	17	15
No. of projects failed	0	2	2	5[b]	4	6
% of projects failed	0	10	10	24	19	29
(b) RC-4D						
Avg. rating of projects	73	66	62	57	61	59
Remaining in service:						
No. of projects rated	12	8	8	5	5	4
No. of projects failed	1	5	5	8	8	9
% of projects failed	8	38	38	62	62	69

[a]See footnote a, Table 5.
[b]Three of the projects which rated below 50 during 1962 rated above 50 in 1963 under revised rating system.

about 3 miles of each type of bitumen placed, all by the same contractor. For all practical purposes, traffic intensity is the same on all sections. When these test sections were placed, there was no significant difference in the appearance of any of the sections. This condition still existed in 1967. Condition ratings have been practically identical for each section during the entire life of these sections. They have all dropped

TABLE 8

CHIP SEALS CONSTRUCTED IN 1959

Item	Year Rated					
	1959	1960	1961	1962	1963[a]	1964
Age	0	1	2	3	4	5
(a) RC-4						
Avg. rating of projects	93	79	75	66	70	68
Remaining in service:						
No. of projects rated	12	11	11	11	11	10
No. of projects failed	0	1	1	1	1	2
% of projects failed	0	8	8	8	8	17
(b) RC-4D						
Avg. rating of projects	91	75	70	64	65	66
Remaining in service:						
No. projects rated	24	24	23	21	24	19
No. of projects failed	0	0	1	3[b]	0	5
% of projects failed	0	0	4	12	0	21
(c) RS-2						
Avg. rating of projects	88	69	63	59	60	57
Remaining in service:						
No. of projects rated	14	14	14	8	8	8
No. of projects failed	2	2	2	8[c]	8	8
% of projects failed	12	12	12	50	50	50

[a]See footnote a, Table 5.
[b]The three projects which rated below 50 during 1962 all rated above 50 in 1963 under revised rating system.
[c]One of the projects which rated below 50 during 1962 rated above 50 in 1963 under revised rating system.

TABLE 9

SUMMARY OF DATA OF ALL SEAL COATS BY AGE

Age (yr)	No. of Projects Constructed			No. of Projects Failed			% of Projects Failed			Avg. Rating of Projects Remaining in Service		
	RC-4	RC-4D	RS-2	RC-4	RC-4D	RS-2	RC-4	RC-4D	RS-2	RC-4	RC-4D	RS-2
0	12	24	16	0	0	2	0	0	12	93	91	88
1	33	37	16	1	1	2	3	3	12	81	74	69
2	41	47	16	3	6	2	7	13	12	78	71	63
3	44	59	16	3	8	8	7	14	50	72	68	59
4	44	59	16	6	9	8	14	15	50	69	65	60
5	44	59	16	7	18	8	16	30	50	68	65	57
6	32	35		8	16		25	46		67	61	
7	11	22		2	8		18	36		70	62	
8	3	12		0	3		0	25		65	62	

TABLE 10

DESCRIPTION OF PROJECTS

Project No.	Approximate Length	Year Built	Type of Oil	Highway Number	Location Description
1	10.856	1959	RS-2	85	From Belle Fourche North
2	18.672	1958	RC-4	79	Jct. 212 to Jct. 24
3	6.547	1956	RC-4D	14	Whitewood to Jct. 85
4	18.616	1958	RC-4D	14A	Cheyenne Crossing to Spearfish
5	16.098	1959	RS-2	85	Cheyenne Crossing to Wyoming Line
6	6.495	1959	RS-2	85	Jct. 14A to Jct. 14
7	19.615	1956	RC-4D	14-16	Rapid City to New Underwood
8	10.817	1959	RS-2	16-385	Hill City to Custer
9	27.146	1958	RC-4D	16	Custer to Wyoming Line
10	12.994	1959	RS-2	385	Sanator to Wind Cave National Park
11	6.795	1958	RC-4D	385	Hot Springs to Wind Cave National Park
12	30.695	1959	RS-2	18	Edgemont thru Hot Springs to Jct. 79
13	12.121	1959	RS-2	18	Oelrichs to East County Line
14	10.142	1959	RC-4D	14-16	East of Cheyenne River to Wall
15	10.089	1958	RC-4	14-16	Quinn to Cottonwood
16	15.623	1959	RC-4D	14	South County Line thru Philip to Powell Jct.
17	4.891	1959	RC-4D	14-16	Wall to Quinn
18	7.528	1959	RC-4D	14	Cottonwood to North County Line
19	15.813	1958	RC-4D	16	Kadoka to East of Belvidere
20	7.033	1957	RC-4D	18	From Martin West
21	14.173	1957	RC-4D	18	Jct. 73 to East County Line
22	24.575	1957	RC-4D	14	West of Missouri River
23	10.799	1958	RC-4D	16	Murdo to Draper
24	11.043	1956	RC-4D	183	From Presho South
25	4.143	1958	RC-4D	16	Oacoma to Missouri River
26	7.928	1958	RC-4	18	Winner to Jct. 183
27	11.243	1959	RC-4D	18	Winner to Colome
28	8.790	1958	RC-4	18	Jct. 183 to Carter
29					Not constructed in 1959
30	17.179	1957	RC-4D	18	Fairfax to Fort Randall Dam
31	7.036	1959	RC-4D	83	Herreid to North Dakota Line
32	8.006	1959	RC-4	83	Herreid to Mound City
33	0.996	1958	RC-4	County	Hosmer East
34	8.659	1956	RC-4D	12	Aberdeen to West County Line
35	18.467	1959	RC-4D	83	Selby South to South Jct. 20
36	0.499	1958	RC-4	County	In Ipswich
37	15.552	1956	RC-4D	12	Ipswich to East County Line
38	14.990	1959	RC-4D	281	Aberdeen to South County Line
39	16.121	1959	RC-4D	83	South Jct. to Jct. 212
40	19.132	1959	RC-4	212	Gettysburg to East County Line
41	13.104	1959	RC-4	212	West County Line to Burkmere
42	15.288	1956	RC-4D	212	Orient Corner to East County Line
43					Not constructed in 1959
44	7.107	1959	RC-4D	212	West of Jct. 83 to Gettysburg
45	40.096	1956	RC-4D	83	Jct. 212 to Jct. 14
46	6.961	1956	RC-4D	14	Highmore to East County Line
47	25.500	1959	RC-4D	45	Miller North to Jct. 212
48	16.875	1956	RC-4D	281	Redfield to South County Line
49	12.489	1958	RC-4D	281	North Beadle County Line South to Jct. 14
50	20.932	1956	RC-4D	14	Pierre to East Blunt

TABLE 10 (Continued)

DESCRIPTION OF PROJECTS

Project No.	Approximate Length	Year Built	Type of Oil	Highway Number	Location Description
51	15.638	1956	RC-4D	14	West Hand County Line to Miller
52	14.472	1959	RS-2	14	Miller to Wessington
53	14.493	1959	RC-4D	14	Wessington to Wolsey
54	6.049	1958	RC-4	County	Jct. 281 thru Virgil
55	7.504	1958	RC-4	47	South Jct. 34 South
56	4.002	1958	RC-4	County	Jct. 281 to Alpena
57	31.096	1959	RS-2	16	Chamberlain to East County Line
58	0.539	1957	RC-4	County	In Plankinton
59	9.474	1957	RC-4	County	Mount Vernon North
60	4.758	1959	RC-4	County	From Platte West
61	21.413	1959	RC-4D	50	Platte to South of Geddes
62	8.864	1957	RC-4D	281	Corsica to Armour
63	4.694	1957	RC-4	County	South of Platte
64	8.328	1957	RC-4D	50	West of Lake Andes
65	5.025	1958	RC-4	County	North of Lake Andes
66	2.608	1959	RS-2	37	South of Tripp
67	6.938	1958	RC-4	18	South of Lake Andes
68					Not constructed in 1959
69	4.972	1957	RC-4	County	North of Wagner
70	17.007	1957	RC-4D	35	Jct. 18 South to Jct. 50
71	0.695	1956	RC-4D	281	North Main and 8th Ave. in Aberdeen
72	5.954	1958	RC-4D	12	West of Jct. 37
73	24.002	1958	RC-4	81	Hammer East and North to North Dakota Line
74	13.218	1957	RC-4D	81	Hammer South to Jct. 10
75	4.004	1959	RC-4	12	West Roberts County Line East thru Ortley
76	12.834	1959	RC-4D	12	Near West Jct. 81 thru Summit to near Marvin
77	13.775	1959	RC-4	12	Near Marvin to Milbank
78	2.494	1959	RC-4D	12	Minnesota Line West
79	12.270	1959	RC-4	25	Webster to South County Line
80	14.989	1959	RC-4	12	Webster to East County Line
81	11.493	1958	RC-4D	81	Jct. 12 to South County Line
82	11.667	1959	RC-4D	77	South of Milbank
83					Not constructed in 1959
84	8.016	1959	RC-4D	81	North of Watertown to Waverly Road
85	27.021	1959	RC-4D	212	Redfield to East County Line
86	6.086	1956	RC-4	County	North and South of Broadland
87	17.754	1958	RC-4	212	Watertown to West County Line
88	6.075	1959	RC-4D	81	Watertown to South County Line
89	9.920	1959	RS-2	212	U.S. 77 to West Deuel County Line
90	18.847	1959	RS-2	77	Clear Lake to South County Line
91	5.010	1959	RS-2	37	Huron North
92	14.855	1956	RC-4D	14	DeSmet to West Kingsbury County Line
93	4.367	1957	RC-4	County	West and North of Hayti
94	9.541	1958	RC-4D	37	Huron to South County Line
95					Not constructed in 1959
96	12.029	1959	RC-4D	34	Madison to West County Line
97	8.848	1959	RS-2	81	South Jct. 14 to South County Line
98	15.494	1958	RC-4D	37-34	East of Woonsocket and North to County Line
99	17.182	1959	RS-2	34	West of Forestburg to East County Line
100	8.985	1957	RC-4	County	Flandreau West of Jct. 77
101	11.191	1959	RC-4D	37	Jct. 34 to South County Line
102	1.491	1958	RC-4	County	Farmer South to 38
103	20.082	1958	RC-4	81	From Salem North to Jct. 34
104	7.240	1959	RC-4	County	North of Montrose
105	10.549	1958	RC-4	County	Dell Rapids West to North of Colton
106	7.075	1959	RC-4	County	Alexandria South to Jct. 42
107	4.118	1958	RC-4	County	Emery South to Jct. 42
108	2.991	1959	RC-4	County	Jct. 19 West to County Line toward Montrose
109	7.699	1958	RC-4	County	Hartford North
110	14.919	1959	RS-2	18	Tripp to Olivet
111	13.267	1957	RC-4D	81	Freeman to South County Line
112	3.949	1959	RC-4	County	Davis South in Turner County
113	5.506	1957	RC-4	County	Canton North
114	5.407	1956	RC-4	County	Centerville toward Hooker in Turner County
115	6.971	1958	RC-4	County	Jct. 77 East to Norway Center
116	5.941	1957	RC-4D	81	South of North County Line to Jct. 46
117	15.995	1958	RC-4	46	Beresford to Iowa Line
118	21.153	1959	RC-4D	19	Vermillion to North County Line
119					Constructed in 1955
120	8.260	1958	RC-4D	12	East of Aberdeen
121					Test Section
122	27.037	1958	RC-4D	12	Webster to West County Line
123	14.320	1958	RC-4	212	Tunerville Jct. to Minnesota Line
124	14.180	1956	RC-4	18	East of Martin to Jct. 73
125	2.095	1959	RC-4D	19A	Jct. 46 North to Centerville
126	8.994	1959	RC-4D	18	Colome East to East County Line
127	12.953	1957	RC-4	10	West of Eureka

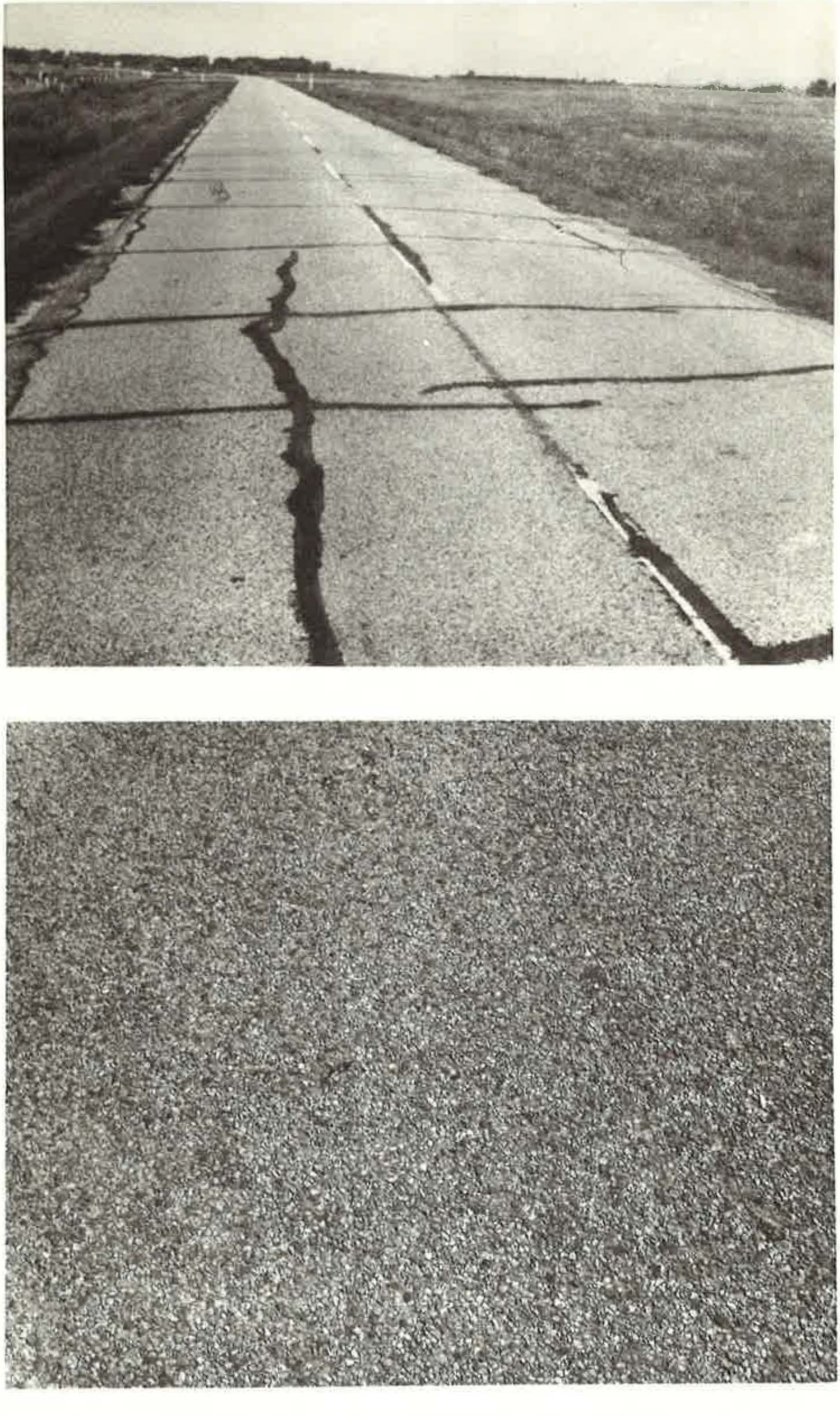

Figure 1. RC-4 with no-strip additive, Aug. 10, 1965.

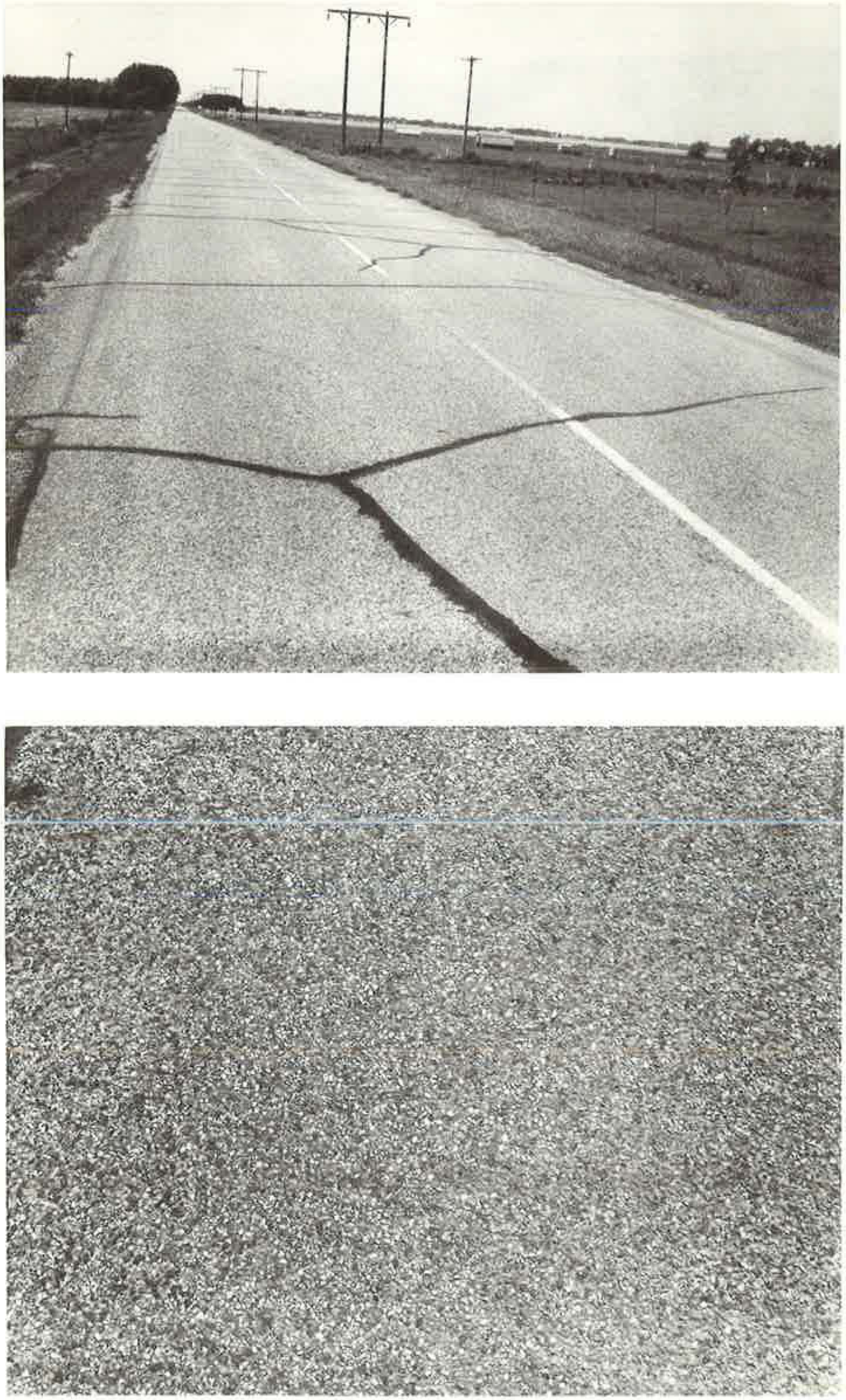

Figure 2. RC-4 with neoprene rubber, Aug. 10, 1965.

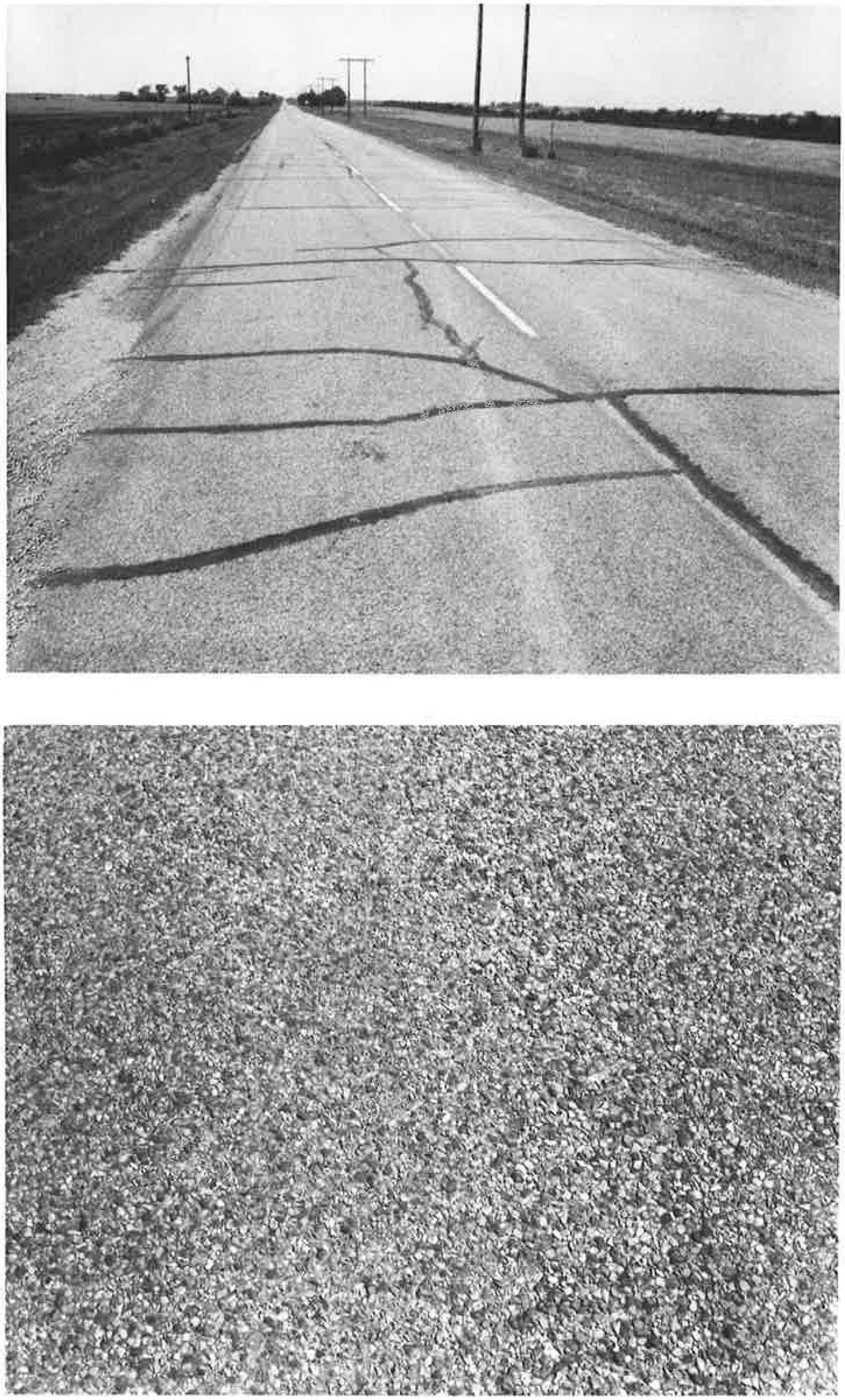

Figure 3. RC-4 with natural rubber, Aug. 10, 1965.

Figure 4. RC-4 with styrene-butadiene rubber, Aug. 10, 1965.

Figure 5. RC-4 with no-strip additive, June 28, 1967.

Figure 6. RC-4 with neoprene rubber, June 28, 1967.

Figure 7. RC-4 with natural rubber, June 28, 1967.

Figure 8. RC-4 with styrene-butadiene rubber, June 28, 1967.

in rating gradually over the years but no significant differences between the four types of bitumen have been noticeable. Figures 1 through 4 are general views and close-ups of each of the surfaces. These photographs were taken on August 10, 1965, when the test sections were seven years old they looked much the same in 1967 at an age of nine years (Figures 5 through 8).

CONCLUSIONS

1. Chip seal coats constructed with plain RC-4 performed better on South Dakota highways than seal coats constructed with rubberized asphalt. Tables 5 through 8 indicate that the percent of projects which failed at any given age is greater among rubberized projects than nonrubberized. The average rating value of those projects remaining in service was higher for those projects constructed with RC-4 than those which included rubber. Table 9 also illustrates these relationships for all projects regardless of year constructed.
2. From Tables 8 and 9, it is obvious that RS-2 emulsion seal coats did not perform as well as either the plain RC-4 or the rubberized seals. This is true both in the percent of projects which failed at any given age and in the average rating value of those projects remaining in service.
3. The differences between the ratings assigned to individual projects constructed with any given type of oil are relatively large (Tables 1 through 4). The difference between the average rating of all projects constructed with RS-2, those constructed with RC-4 and the rubberized asphalt at any given age was relatively small (Table 9). It may be concluded that factors other than oil type influence the rating of any individual seal coat project. Such factors as construction control, weather during construction, type and condition of equipment, and the experience of contractor personnel are variables which may have a profound effect on the rating assigned any particular seal coat project.
4. Figure 9 indicates that under South Dakota climatic conditions a chip seal will seal most cracks during the summer in which it was built. It is apparent, however, that during the first winter contraction causes the cracks to reappear and at the age of one year the chip seal coat is not effective in sealing cracks. None of the three types of asphalt used in this study satisfactorily sealed cracks at the age of one year.
5. The life expectancy of a chip seal coat is not determined exclusively by the type of asphalt used. Under South Dakota traffic conditions the length of life of a properly constructed chip seal depends to a large extent on the quality of the highway over which it is placed. Many chip seals which are properly constructed are buried under patches within 5 to 8 years after construction due to failures in the underlying structure. Several seals placed on good highways are still extremely serviceable after 8 to 11 years.

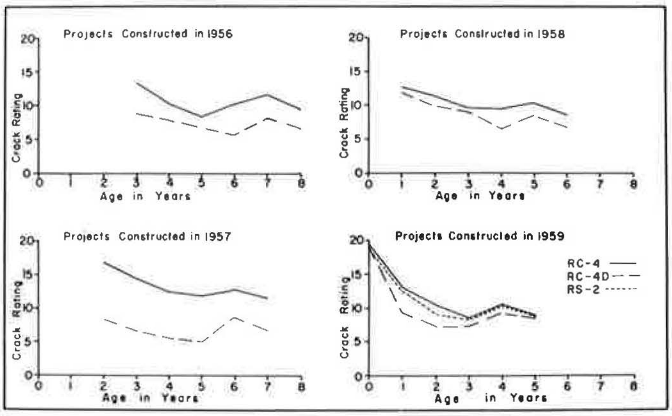

Figure 9. Average crack rating of seals constructed with various asphalts at various ages.

RECOMMENDATIONS

The following recommendations are based on experience obtained in rating and observing chip seals throughout the years during which this study was being conducted. Although these recommendations are not necessarily based on the data presented in this report, they are based on experience gained during this study and are thought to be of sufficient importance to be given consideration in future seal coat construction.

1. Chip seals should be rated after one year of service. Traffic and climate have subjected the seal to their influence but subgrade weaknesses have not generally required extensive repairs at this age. A seal performing well after one year will nearly always perform well for a number of years.
2. Old roads or weak roads with a history of high maintenance requirements should never be sealed. On this type of road patching will soon cover the seal coat. It is felt that the money could be more wisely spent in reconstruction on roads of this type.
3. Before sealing any road, know for what reasons the seal is being applied. Do not seal out of habit or because it is "company policy." Ask the question: "Will a seal coat correct the deficiencies in the pavement?"
4. Follow specifications.
5. Avoid sealing in cold or wet weather.
6. Keep distributor functioning properly. Keep nozzles clean, prevent them from plugging, maintain proper height above ground, etc.
7. Prevent overlap of oil shots both transversely and longitudinally.
8. Maintain uniform application rates.
9. Use full-time inspectors with training and experience in chip sealing operations.
10. The use of additives is not recommended until definite benefits have been demonstrated in both laboratory and field tests which justify the additional expenditure involved.

REFERENCES

1. Rubberized Asphalt Tested in South Dakota. Roads and Streets, p. 192, February 1959.
2. Seal Coat Evaluation Procedure. Highway Research News, No. 24, p. 40-52, Summer 1966.

Control of Binder Distribution in Bituminous Surface Treatments

DAVID C. MAHONE and STEPHEN N. RUNKLE, Virginia Highway Research Council

The control of binder distribution has long been a factor of major concern to those responsible for bituminous surface treatment work. Experimental work was conducted during the summers of 1964 and 1965 in Virginia with the trough test, cup test, and cotton pad test in order to determine and define reasonable variability from the specified mean binder application rate both transversely and longitudinally; and to provide a method of test to be used in end result specifications for the control of binder application.

The investigation was limited in that most of the work was performed on state secondary roads, and the distributors, most of which were state-owned, were those in use during 1964 and 1965.

It was found that the trough test and cup test were not suited for field use and, more importantly, could not be used for enforcement of end result specifications. The cotton pad test was found to be suited for field use, and can be used in end result specifications.

It was found that both the longitudinal and transverse consistency could be checked periodically by running a cotton pad test. Also, based on the tests made, it was determined that for the equipment tested the longitudinal variation should be no more than ± 8 percent from the intended rate, and that the transverse variation as indicated by one cotton pad test should be 8 percent coefficient of variation or less. (It was decided the coefficient of variation was the best measure of transverse variability.)

THE very best materials and design can go into a bituminous surface treatment and the results still be unsatisfactory unless the binder is evenly distributed to the proper depth. Poor control of the binder application could result in streaking, excessive loss of cover stone, bleeding, and in fact, almost any of the types of failure common in surface treatments. Since this control is of utmost importance in every surface treatment regardless of the physical characteristics of the aggregate or binder, the natural elements, or the traffic that the road will carry, it is a factor of major concern to those responsible for surface treatments. Consequently, this investigation was devised and included in the 5-yr bituminous surface treatment study being conducted in Virginia in an attempt to improve the quality and service life of such treatments.

PURPOSE AND SCOPE

Specifically, the purpose of this investigation was twofold: (a) to determine and define reasonable variability from the specified mean binder application rate both transversely and longitudinally; and (b) to provide a method of test to be used in end result specifications with regard to binder application.

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The scope was limited in that the findings are based on the performance of binder distributors in use in Virginia during 1964 and 1965 (most of which were state-owned) and three types of asphalt: CAE-2, RC-2, and AP-00.

To evaluate the performance of the machines, three test methods—the cup test, trough test, and cotton pad test—were used. However, only the last lends itself to end result specifications.

The experimental work was conducted during the summers of 1964 and 1965 on state roads, mostly secondary, through the cooperation of the Central Office and the field personnel of the Department of Highways. During 1964 a total of 10 bituminous distributors were investigated for distribution characteristics, and during 1965 14 more were tested in an attempt to evaluate the usefulness of the cotton pad method.

TEST PROCEDURES

The cup test and trough test require that the distributor be stationary, whereas the cotton pad test is employed during actual operation.

Distributor Stationary

Two test methods were used to evaluate the lateral distribution of the spray bar while the distributor was stationary: one was a measure of the quantity discharge from each nozzle; the other was a measure of the distribution pattern across the bar. The quantity of asphalt is influenced by the nozzle discharge only, whereas the distribution pattern is influenced by both the nozzle discharge and nozzle angle.

The amount of binder discharge from the individual nozzles was measured by placing quart paper containers under each nozzle and spraying into them simultaneously. A rack was used to keep the cups from turning over (Fig. 1).

The second stationary method of checking the spray bar was a check on the distribution pattern of the binder. As previously mentioned, this is influenced by both nozzle discharge and nozzle angle. Figure 2 shows the 12-ft metal trough used in this test. It is divided into forty-eight 3-in. sections, 7 in. deep, and it is 18 in. wide at the top and 9 in. at the bottom. The trough, which was modeled after a larger unit used in Pretoria, Transvaal, South Africa, for calibrating distributor spray nozzles and bars, can be disassembled into two equal units for transportation (2, 3).

Obviously the trough test and cup test are not useful in end result specifications since they are tests on the machines and not the work. It was hoped, however, that they would be useful in determining what standard of work should be expected from the equipment available.

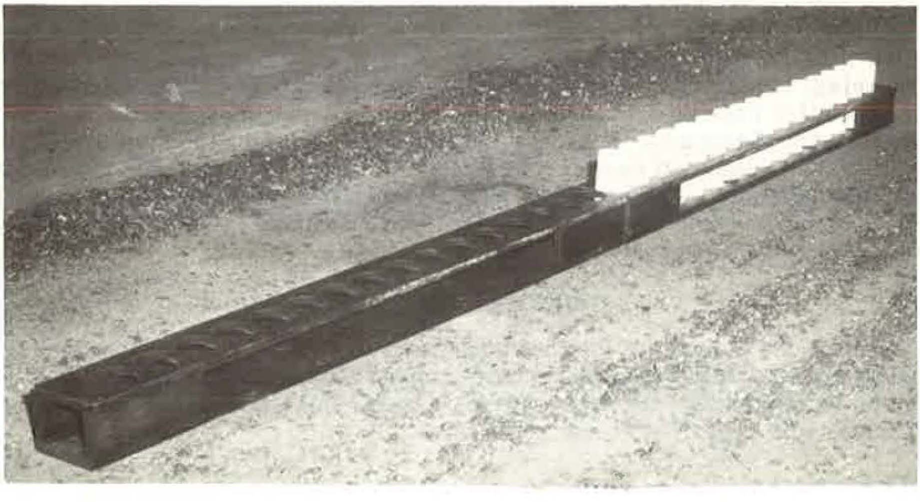

Figure 1. Paper cup arrangement for measuring nozzle discharge.

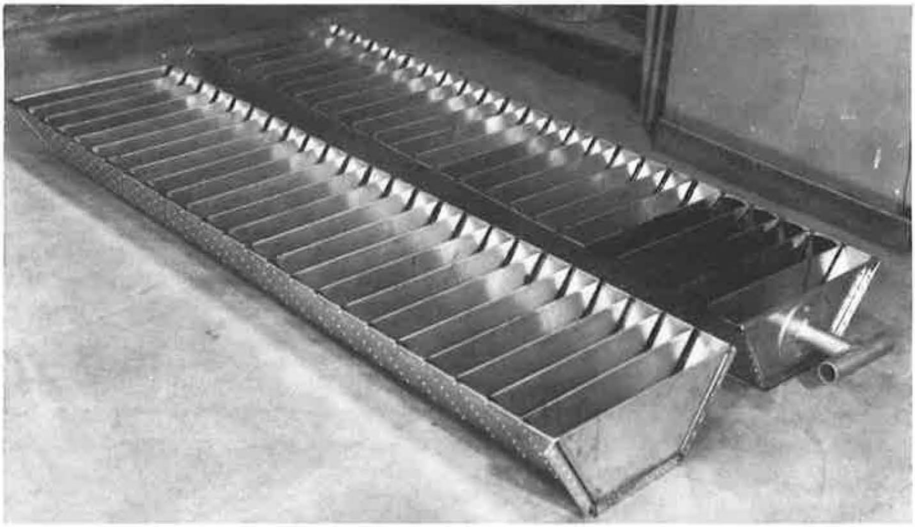

Figure 2. Calibration trough, disassembled.

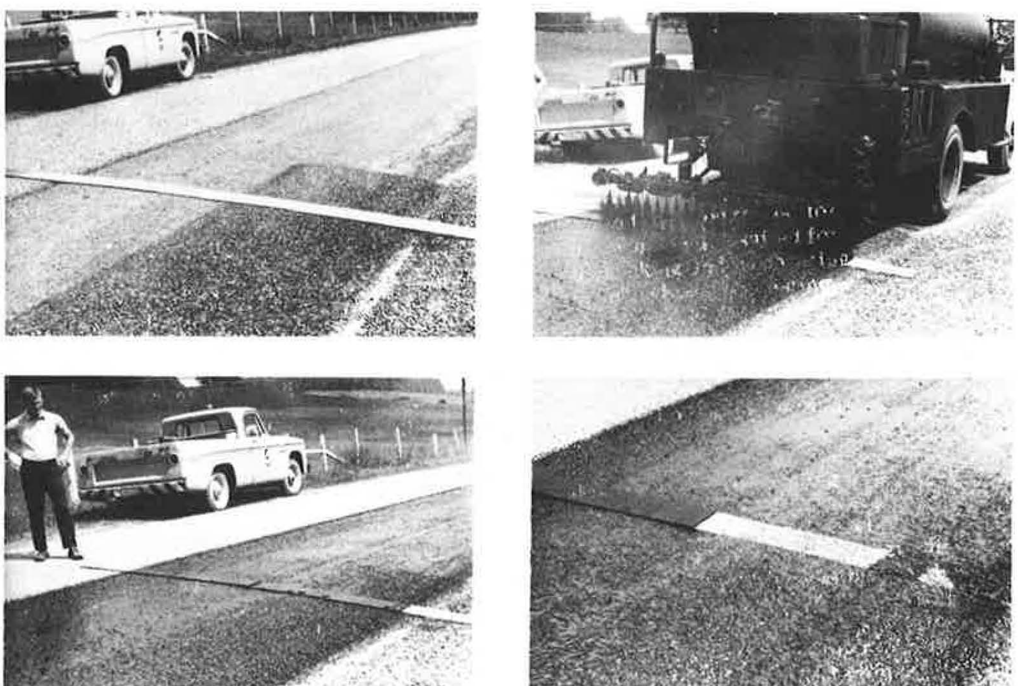

Figure 3. Cotton pad test strips being used to measure lateral distribution.

Distributor Operating

The lateral and longitudinal distribution of the spray bar may be evaluated while the distributor is operating, i.e., actually spraying binder onto a road. The procedure, developed by the California Division of Highways, is to glue 4-in. wide cotton pads to sheets of paper, which are in turn placed on metal sheets. The sheets are placed on the roadway just ahead of the distributor (Fig. 3). The term "test" is used to identify this operation, while the term "sample" refers to one 4 by 8-in. pad. The cotton prevents the binder from flowing until the sample can be weighed. In each test the end pads (the last pad on each end that appears to be covered) are discarded.

The major advantage of the cotton pad test is that it tests for the amount of binder actually being applied on the roadway. Thus, the transverse and longitudinal variability of binder on the road can be measured.

PRELIMINARY INVESTIGATION

During 1964, ten asphalt distributors were tested in order to determine the usefulness of the three tests. It was found that neither the trough test nor cup test was suitable for field use. The following difficulties were encountered with these two methods.

1. Although the binder was circulated through the bar for 5 minutes before shooting, it was noted that the nozzles did not always come on at full force simultaneously. This could be due in part to clogged nozzles, or to all sections of the bar failing to activate simultaneously. However, another possible cause, which was suggested by leakage on many spray bars, was worn parts.
2. It is almost impossible to find a level place in the field and on most distributors the header bars are not truly horizontal extensions of the spray bar. Therefore, when the highest nozzles were low enough to insure shooting into a specific cup, other points along the bar were resting on cups. This interfered with the activation of the spray bar and also bent cups, causing loss of binder, and at times, turned cups partially over.
3. The difficulty in finding level testing sites and the nonhorizontal extensions of the spray bars also affected the trough test since the height and angle of the spray fan over the trough affected the distribution pattern.
4. The relatively small size of the cups and trough sections resulted in a very short shooting time, thus exaggerating any error due to the difficulties mentioned previously.
5. Both processes were time consuming and tied the distributor up for longer periods than desirable. This of course held up the entire surface treating operation.
6. Since there was no way to recover the binder shot into the trough or cups, much was wasted.

Although the tests were judged to be unsatisfactory for field use, in our opinion they could be most useful in checking and calibrating distributors at a specially prepared central location.

It was found that the cotton pad test was much more suitable for field work than either of the other methods. Most of the problems encountered with this test have the common origin of motion. Because the distributor is moving it is difficult to determine which spray fans are covering specific pads. Since the pads have to be removed from the sheets and transported to the weighing area there is a tendency for the binder to flow from pad to pad. For this reason it is recommended that the test not be used when the amount of binder being applied is greater than 0.40 gsy.

Since the test was performed after the distributor had been operating for a while the problem of uneven bar activation did not exist. Any variability in the transverse distribution could be assumed to be a continuing condition.

Despite the problems associated with motion, it was decided that the cotton pad test could be used as a method of test for end result specifications with regard to binder application. It was also decided more field work was necessary during 1965 to determine reasonable transverse and longitudinal variability.

COTTON PAD TESTS—1965 INVESTIGATION

Specifically, the following were to be determined during 1965: (a) the best method of expressing variability, and with regard to transverse distribution, the amount of variability that should be expected in the measurement from test to test without any adjustments to the equipment; and (b) based on the work in 1964 and 1965, the allowed standards for transverse and longitudinal variation, and the number of tests required to insure that the standards are met under various assumptions.

TABLE 1

LARGEST SAMPLE SIZE REQUIRED TO DETERMINE MEAN ±8 PERCENT AT A 95 PERCENT CONFIDENCE LEVEL

Machine No.	Largest Sample Size[a]	Range for All Tests
1	8	6-8
2	10	5-10
3	7	5-7
4	7	2-7
5	9	4-9
6	7	3-7
7	3	1-3
8	7	3-7
9	8	3-8
10	17	9-17
11	9	3-9
12	7	2-7
13	13	4-13
14	13	2-13

[a] Largest sample size required of 7 tests made on each machine.

Fourteen distributors were used during the summer. With the first 7 distributors, 7 sheets of cotton pads were placed side by side in as close an area as possible. As soon as the distributor had passed over all 7 sheets, the individual cotton pads were removed and weighed immediately. With the last 7 distributors, the tests were taken in approximately 200-ft intervals in order to help determine how consistent the longitudinal application should be.

Longitudinal Variation

In all the tests (Table 1), it was rarely necessary to have a sample size greater than 10 cotton pads in order to predict the mean within ±8 percent at a 95 percent confidence level (6 out of 96 tests and 4 times with the same distributor). Thus, the longitudinal consistency of binder application could be checked by randomly selecting and weighing 10 pads from each test made. In this way a check could be made as to whether the actual average amount being applied is within ±8 percent of the intended application.

Perhaps an even better approach to checking longitudinal consistency only is simply to weigh at one time all of the pads covered by binder. If this was manageable, it would reduce the time required to complete the test and be more accurate than weighing 10 pads. No attempts were made to take the test in this manner during the field work, but it certainly seems to be feasible.

It should be remembered also that the sample size of 10 pads is only tentative in any case. If adjustments are made to a distributor and they reduce the transverse variability, the sample size necessary for predicting the average will decrease.

The results also indicate that the sample sizes required are larger with RC-2 (cutback) and CAE-2 (emulsion) than with AP-00 (200-300 penetration). This is true probably because RC-2 and CAE-2 are usually applied in greater quantities than is AP-00, and therefore have a greater tendency to flow from pad to pad during handling and thus cause greater variation. As mentioned earlier, the test is not recommended when the amount applied is greater than 0.40 gsy.

Transverse Variation

Since normally the absolute variation would increase as the test mean increases and decrease as the test mean decreases, it was decided that the best measure of transverse variation was the coefficient of variation, which is the standard deviation of the test expressed as a percentage of the test mean ($V = 100\ s/\overline{X}$). By using the variation coefficient as the measure of variability, the relative variability between tests is determined and is therefore uninfluenced by the magnitude of the test mean. Table 2 gives the amount of variation to be expected in the variation coefficients between tests for the same machine when no adjustments are made between tests.

For only three machines was the standard deviation of the variation coefficients more than 2.0 percent (machines 11, 13, and 14), and in two of these (machines 13 and 14) the tests were taken on roads where it was difficult to find level test sites, which perhaps caused a greater coefficient of variation in some of the tests. Judging from the results, it seems safe to say that the normal standard deviation to be expected with level test sites usually would not exceed 2.0 percent.

The standard deviation determines the position of the control limit in relation to the standard in testing for compliance with standards. For instance, if a standard of 8 percent was desired and only one test was taken in various intervals then the control limit to determine if the standard was met would be above the 8 percent standard, the amount

TABLE 2

COEFFICIENTS OF VARIATION

Test	Tests Taken Together							Tests Taken in 200-Ft Intervals						
	Mach. 1 AP-00	Mach. 2 AP-00	Mach. 3 AP-00	Mach. 4 CAE-2	Mach. 5 CAE-2	Mach. 6 CAE-2	Mach. 7 RC-2	Mach. 8 AP-00	Mach. 9 AP-00	Mach. 10 CAE-2	Mach. 11 CAE-2	Mach. 12 RC-2	Mach. 13 RC-2	Mach. 14 RC-2
1	10.0%	12.7%	9.5%	6.2%	8.1%	9.4%	6.6%	7.5%	10.9%	12.0%	6.6%	9.6%	14.4%	14.1%
2	10.4	10.4	9.4	6.5	10.2	6.4	7.3	10.7	10.1	11.9	11.9	6.0	—	5.8
3	9.8	10.7	10.2	7.3	8.7	7.6	7.4	8.9	8.8	12.3	7.7	9.6	11.6	11.7
4	10.2	10.8	8.7	9.7	11.6	7.3	5.8	8.9	8.9	12.9	6.3	10.8	9.0	9.2
5	10.1	9.2	9.6	10.4	9.8	7.0	6.8	8.5	7.6	13.4	7.2	10.1	8.0	7.9
6	10.4	10.3	9.6	7.6	11.0	10.4	4.6	8.5	7.2	16.4	4.8	10.9	8.5	8.7
7	11.4	—	10.2	7.4	9.9	8.6	6.7	9.0	8.4	13.9	7.1	8.1	8.5	7.7
Average	10.3%	10.7%	9.7%	7.9%	9.9%	8.1%	6.5%	8.9%	8.8%	13.3%	7.4%	9.3%	10.0%	9.3%
Standard deviation	0.5	0.11	0.5	1.6	1.2	1.4	1.0	1.0	1.3	1.6	2.2	1.7	2.5	2.8

TABLE 3

COEFFICIENTS OF VARIATION BY ASPHALT TYPE

Asphalt Type	AP-00	CAE-2	RC-2
Average	0.9	1.6	2.0
Range	0.5-1.3	1.3-2.2	1.0-2.8

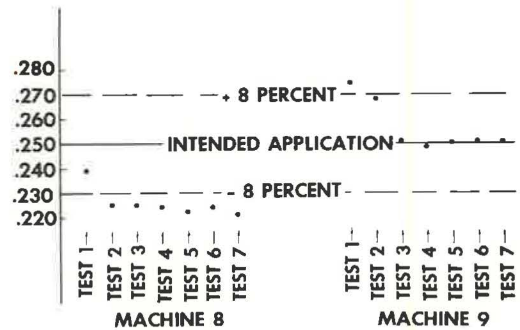

Figure 4. Longitudinal variability.

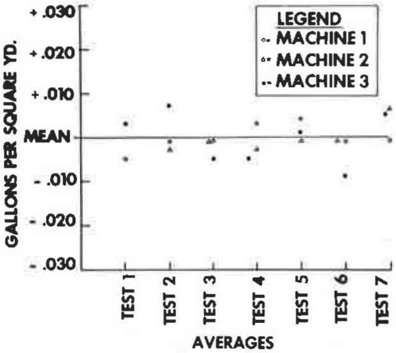

Figure 5. AP-00 asphalts (machines 1, 2, and 3).

depending on what confidence of not rejecting a good job is desired. Plus two standard deviations (4 percent) would set the control limit at 12 percent and mean that only about 2.5 percent of the time would a test show that the standard is not met when actually it is.

The variation is less for AP-00 than for the other two types of asphalt. The comparisons of the standard deviations for the different asphalts are given in Table 3. On the basis of these figures, it may be wise to consider the type of asphalt both in setting a standard for transverse variation and in measuring to determine if the standard is being met.

In addition to the determination of transverse variation by testing the overall variability of a test, an analysis was made to determine if the means of one pad (4 in. wide) and if 1-ft transverse sections could be determined within ± 10 percent at a 95 percent confidence level with a reasonable number of tests. This was done by determining the percent deviation of each pad from its test mean and then comparing corresponding pads on separate tests. It was found that for 1-pad wide sections 4 to 9 tests would be required, and for 1-ft wide sections only 2 tests would be required. This method of analysis is possible but requires more computation and offers no advantage over testing for overall variation. Although it is possible to determine what pads of the test are high, it is difficult to determine what pads are covered by what nozzle and which nozzles need adjusting. Also, for end product specifications it makes more sense to deal with overall variation. Therefore the analysis is not presented.

Standard Longitudinal Variability

Not many data are available for determining the standard for longitudinal consistency; however, the available data indicate that the mean amount being applied should be within at least ± 8 percent of the intended application rate. Also, some 1964 data which are not presented substantiate this standard. The variation of the test means for the two sets of tests taken in 200-ft intervals in which AP-00 was used is shown in Figure 4.

Based on these tests, the ± 8 percent standard appears to be very reasonable. Although for machine 8 the application amount is 10 percent below the intended application, the consistency from test to test is extremely close. In our opinion the standard could be lower, and certainly should be no more than ± 8 percent.

Meeting the ± 8 percent standard for longitudinal consistency means only that the test mean is within 8 percent of the intended rate and indicates nothing about the transverse variability. Thus the standard for longitudinal consistency could be met and the transverse distribution be extremely variable.

Seven machines were tested in 200-ft intervals, but only the two shooting AP-00 were considered in determining the longitudinal standard because of evaporation. In Figure 5, the binder is AP-00 and there obviously is no evaporation from the first to the last test; but in Figure 6 with CAE-2 there obviously is evaporation. In using the cotton pad test to check for the amount of binder being applied when those types of binder which are not pure asphalts are used, it is most important to weigh each test immediately or to have a control sample so that the evaporation rate

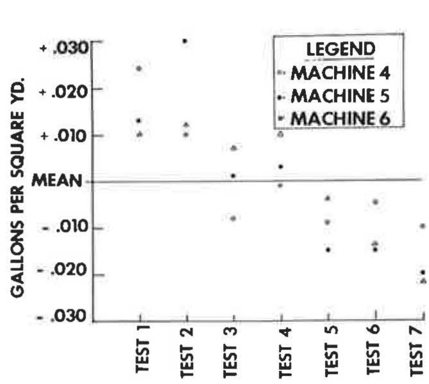

Figure 6. CAE-2 asphalts (machines 4, 5, and 6).

TABLE 4

TRANSVERSE VARIABILITY CONTROL LIMITS
(All Numbers Are Percentage Figures)

(1) Maximum variation coefficient allowable.	10				12				14			
(2) Percent of binder varying more than max. in (1).	0		2.5		0		2.5		0		2.5	
(3) Average variation coefficient, or "standard," necessary for conditions (1) and (2) to be met.	4		6		6		8		8		10	
(4) Chance of not accepting product meeting standard.	0	2.5	0	2.5	0	2.5	0	2.5	0	2.5	0	2.5
(5) Control, or rejection, limit for sample size of one test.	10	8	12	10	12	10	14	12	14	12	16	14
(6) Control, or rejection, limit for sample size of two tests.	8.2	6.8	10.2	8.8	10.2	8.8	12.2	10.8	12.2	10.8	14.2	12.8
(7) Control, or rejection, limit for sample size of four tests.	[illegible]	6	9	8	9	8	11	10	11	10	13	12

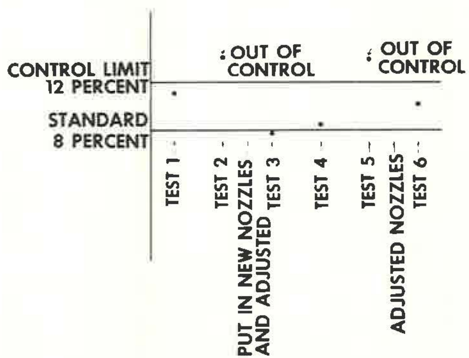

Figure 7. Transverse variation control chart (machine 4—1964).

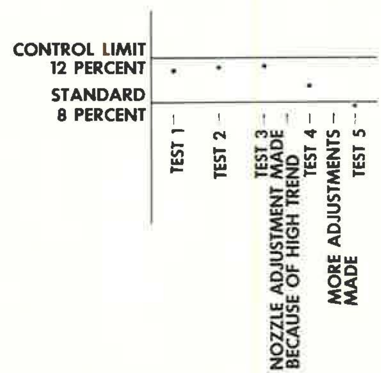

Figure 8. Transverse variation control chart (machine 5—1964).

can be known. With the test, the evaporation problem is one reason why it is advantageous to have to weigh as few pads as possible in checking for longitudinal consistency. Evaporation should not affect the transverse variability, assuming all pads are subject to the same rate of evaporation, since the variation coefficient is the measure used.

Standard Transverse Variability

For transverse variation, the authors believe the standard should be 8 percent variation coefficient considering the results of the tests made during 1964 and 1965. If the standard is met 68 percent of the binder will be within ± 8 percent (one variation coefficient), 95 percent within ± 16 percent (two variation coefficients) and 100 percent within ± 24 percent (3 variation coefficients). For the 96 tests made in 1965 (Table 2) the average variation coefficient was 9.3 percent. No adjustments were made to the machines before any of the tests. The least average variation coefficient was 6.5 percent, and for four machines the variation coefficients were below or near 8.0 percent. It seems reasonable that in all applications the variability could be as low as in these distributors. During 1964 adjustments were made to the distributors between tests. For 4 of the 10 machines on which the nozzle output was judged to be good and adjustments were made to the nozzles, the average variation coefficient was 6.9 percent and the range was 4.5 to 9.6 percent. Based on these tests, an 8 percent variation coefficient certainly appears to be reasonable.

Testing for Compliance With Standards

Each time a cotton pad test is made the mean amount of binder being applied in the road at the test site will be known. Thus, in checking for longitudinal consistency it is a matter of engineering judgment as to how often tests should be made.

For transverse variability the number of tests required in a given interval is determined by the desired assurance that the standard is being met. Table 4 gives some control limits for sample sizes of 1, 2, and 4 tests assuming a standard deviation of 2 percent (Table 1). The control, or rejection, limit decreases as the number of tests increases. Normally in quality control it is desirable to have a sample size larger than one since the control limits can be made tighter. However, for the cotton pad test it may be desirable to use a sample size of one test. If it is used as a control tool, adjustments may be made to the distributor after one test, especially if it is obviously not producing acceptable work; if it is used as an acceptance tool, it is desirable to have as few tests as possible since each test is relatively time consuming.

An example of how work may be controlled or accepted through use of a control or acceptance chart technique is shown in Figure 7. The standard was set at 8 percent average variation coefficient, with 2.5 percent varying more than 12 percent and a 2.5 percent chance of not accepting work meeting the standard (Table 4). The results are from a machine tested during 1964.

The effect of nozzle adjustments and cleanliness of the nozzles is well demonstrated in Figure 7. This machine was more variable than most tested, probably because of worn parts in the spray bar. A machine producing more consistent work is represented in Figure 8. In this case, even though no tests were out of control, the first three showed a tendency to be high. After adjustments were made, the test results improved.

This same type of control chart technique could be used for longitudinal consistency. In Figures 7 and 8, the technique was demonstrated as a control device. It could just as well be used as an acceptance tool, where results out of control would be rejected.

CONCLUSIONS

1. The cup test for determining nozzle output and the trough test for determining the transverse distribution pattern are not suited for field use, but may be extremely useful in checking and calibrating distributors if performed at a specially prepared central testing site.

2. Although few data are available for determining what the standard for longitudinal consistency should be, in the opinion of the authors the measured amount of binder being

applied should be no more than ± 8 percent from the intended application rate and perhaps less.

3. Longitudinal consistency can be checked periodically by running a cotton pad test. For the tests run, rarely was it necessary to weigh more than 10 randomly selected 4-in. pads to determine the mean within ± 8 percent at a 95 percent confidence level. The best approach for checking longitudinal consistency may be to weigh all pads at once, although this was not tried during the field work.

4. In checking longitudinal consistency, care must be taken when binders subject to evaporation are used. Either the test pads must be weighed immediately or a control sample made.

5. The best measure of transverse variability is the coefficient of variation. When this measure is used the standard deviation to be expected is about 2 percent. However, for AP-00 asphalt it is lower, so in setting control or rejection limits it may be wise to consider the type of asphalt being used. (Although the means of various size transverse segments can be determined within ± 10 percent at a 95 percent confidence level, it is our opinion that the best approach to determining transverse variability is by testing the overall test variability.)

6. The standard for transverse variability should be 8 percent coefficient of variation or less based on the data collected in 1964 and 1965.

7. The cotton pad test could be a most useful tool for controlling, and for accepting or rejecting, binder applications with regard to both longitudinal and transverse variability. Although they were not demonstrated in this paper, analytical techniques could be used to reduce greatly the statistical work necessary in using the test and make it more applicable for field use.

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Skid Resistance of Screenings for Seal Coats

ERNEST ZUBE and JOHN SKOG, California Division of Highways

Providing and maintaining a skid-resistant surface is a very important factor in the performance of any highway. All types of pavement surfaces will eventually show some reduction in coefficient of friction values during their service life. This reduction is caused by wear and polish of traffic.

A satisfactory method has been devised for determining in the laboratory the original coefficient of friction of seal coat screenings by use of fabricated test panels and the California skid tester. Friction values obtained from the test panels correlate very well with field test installations.

A laboratory method for wear and polish studies has been developed which shows good correlation with actual service performance of screenings.

ONE of the primary purposes of a screening seal coat is to improve the skid-resistance characteristics of an existing asphalt concrete pavement. It is very important that suitable tests be developed that will provide screenings having a high original friction value and a high degree of resistance to reduction in the friction factor by wear and polish due to traffic.

Maclean and Shergold (1), in their studies on British screening sources, indicated definite differences in wear and polish, and serious reduction in the friction factor after a relatively short service life. This reduction was directly caused by traffic wear and polish since the seal coats remained in excellent condition with full chip retention.

This report presents studies by the California Division of Highways on the development of laboratory tests for measuring the original coefficient of friction value for seal coat screenings, and a laboratory method for determining the amount of wear and polish that may be expected during service life. The various laboratory tests have been correlated with field performance by placing screenings from different California commercial sources at two test locations.

This project is only one phase of a Bureau of Public Roads supported program on skid resistance. Other phases involve the determination of a minimum friction figure for remedial action and methods for raising the skid resistance of existing pavements, such as grooving of the surface. Also planned are studies on wear and polish of portland cement and asphalt concrete surfaces.

MEASUREMENT OF SKID RESISTANCE

The California skid tester used in determining the coefficient of friction of laboratory and field seal coat screening test surfaces has been previously described (2). The present test method, using this equipment, is presented in Appendix A.

The skid tester has been calibrated with the towed cart equipment constructed by R. A. Moyer of the University of California, Institute of Transportation (3). Previous studies by Moyer and others indicated that the skid-resistance value for any given surface approaches a low figure when the brakes are locked on a vehicle having smooth tread tires and traveling at speeds of 50 mph on a wet pavement. Therefore, in the correlation program, the coefficient of friction values obtained from Moyer's unit using

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locked wheels, smooth tires, wet pavement and a speed of 50 mph were compared to our readings obtained under identical operating conditions.

We are presently using a value of 0.25f as the minimum requirement for indicating the need for remedial action. An active program is under way to study the adequacy of this value. The studies to date appear to indicate that the value should possibly be raised to 0.28f on curves and perhaps could be reduced to 0.22f on long tangents. However, in this paper we will consider any screening seal coat surface adequate for skid resistance if the average value is 0.25f or above. This, of course, means that at no time during the service life of the seal coat shall the surface have a lower value than 0.25f. The change in the value during service life is assumed to be caused by traffic wear and polish and not from "bleeding," loss of screenings by displacement or from ice formation.

LABORATORY STUDIES: TESTS FOR ORIGINAL SKID RESISTANCE

Since all screenings will ultimately wear and polish during service life, it is important to purchase materials having the highest possible original coefficient of friction, and maintaining a satisfactory value. Therefore, a method was developed for fabricating laboratory test plates that closely simulate the field wearing surface. Test panels are prepared on 30-lb roofing felt using 0.2 gal/sq yd of penetration or high viscosity emulsion and 20 lb/sq yd of chips. The surface is immediately rolled with a small hand roller and, after curing for 24 to 48 hours, surplus chips are removed by inverting the test specimen. The panel is then heated to 120 to 140 F and rolled again. After cooling to room temperature, the panels are then tested with the California skid resistance tester (2). The friction values for laboratory prepared specimens are compared with the original readings on field test patches (Table 1 and Fig. 1). The results are considered quite good since friction values above 0.40 may be quite variable.

The preparation of the test plates requires considerable time so studies were undertaken to develop a method that would be simple and require only one sample of

TABLE 1

COMPARISON OF ORIGINAL COEFFICIENT OF FRICTION VALUES OF LABORATORY TEST PLATES AND FIELD TEST PATCHES

Code No.	Sample No.	Size	Original Coefficient of Friction (f): Lab Test Plate	Field Test Patch: Auburn	Field Test Patch: Stockton	Field Test Patch: Avg.
1	56-1438	¼ × No. 10	0.42	0.39	0.38	0.39
2	60-4034	5/16 × No. 8	0.43	0.41	0.43	0.42
3	61-507	5/16 × No. 8	0.41	0.39	0.41	0.40
4	61-509	5/16 × No. 8	0.35	0.35	0.40	0.38
5	61-511	⅜ × No. 6	0.38	0.39	0.39	0.39
6	61-516	5/16 × No. 8	0.42	0.41	0.41	0.41
7	61-543	5/16 × No. 8	0.40	0.42	0.43	0.43
8	61-545	5/16 × No. 8'	0.42	0.41	0.43	0.42
9	61-551	5/16 × No. 8	0.41	0.41	0.38	0.40
10	61-552	5/16 × No. 8	0.38	0.42	0.44	0.43
11	61-573	5/16 × No. 8	0.40	0.40	0.42	0.41
12	61-574	5/16 × No. 8	0.40	0.41	0.44	0.43
13	61-586	5/16 × No. 8	0.39	0.40	0.43	0.42
14	61-588	¼ × No. 10	0.41	0.41	0.44	0.43
15	61-590	5/16 × No. 8	0.43	0.41	0.43	0.42
16	61-600	⅜ × No. 6	0.40	0.41	0.41	0.41
17	61-601	¼ × No. 10	0.40	0.42	0.43	0.43
18	61-602	⅜ × No. 6	0.41	0.38	0.42	0.40
19	61-603	¼ × No. 10	0.41	0.41	0.45	0.43
20	61-893	5/16 × No. 8	0.37	0.35	0.41	0.38
21	61-1240	5/16 × No. 8	0.41	0.41	0.40	0.41
Avg.			0.40	0.40	0.42	0.41

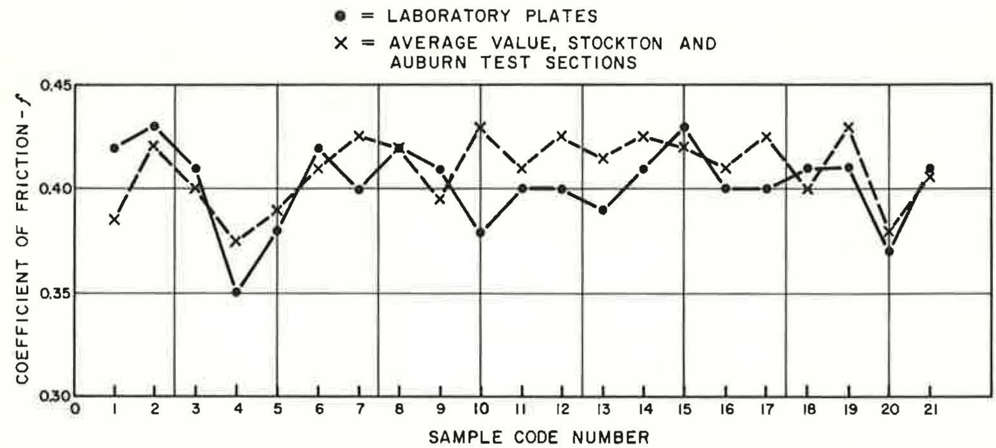


Figure 1. Comparison of original coefficient of friction values of laboratory test plates and field test patches.

screenings. The method finally developed makes use of our centrifuge kerosene equivalent test (CKE), California Method 303, and provides a value for the surface texture and particle shape characteristics of seal coat screenings (Appendix B). Briefly, the sample consists of 100 g ± 1.0 g of washed and dried aggregate passing a No. 3 sieve and retained on a No. 4 sieve. The sample is saturated in kerosene for 10 min and centrifuged for 2 min at 400x gravity. The weight of the sample is then recorded to the nearest 0.1 g. (This operation satisfies the absorption.) The sample is then removed from the centrifuge cup and placed in the CKE test funnel. The funnel is submerged in S.A.E. No. 10 lubricating oil, raised immediately and allowed to drain. The difference in weights after centrifuging and after draining represents a surface factor for the screening sample. This factor, after a specific gravity correction, is designated K_S.

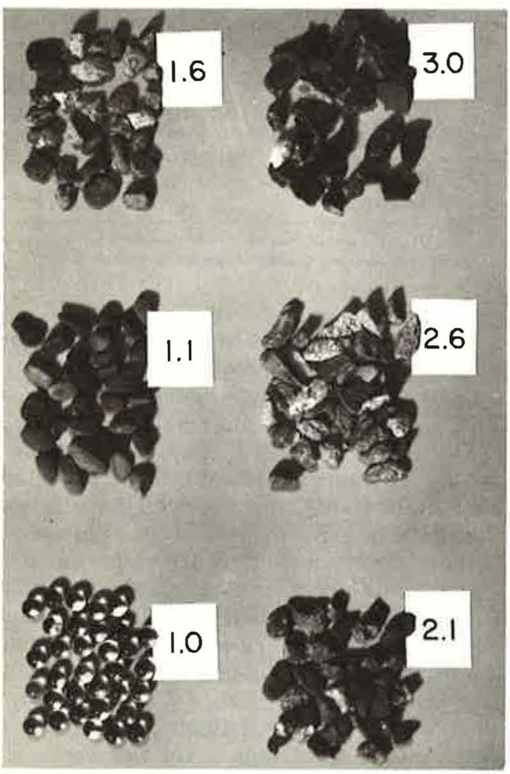


Figure 2. Surface constant K_s values for glass beads and various screenings.

TABLE 2

CHANGE IN K_S VALUE ON ADDITION OF CRUSHED QUARTZ TO GLASS BEADS

Material	K_S Value
Glass beads	1.00
75% Glass beads 25% Crushed quartz	1.39
70% Glass beads 30% Crushed quartz	1.44
50% Glass beads 50% Crushed quartz	1.83
40% Glass beads 60% Crushed quartz	1.93
10% Glass beads 90% Crushed quartz	2.28

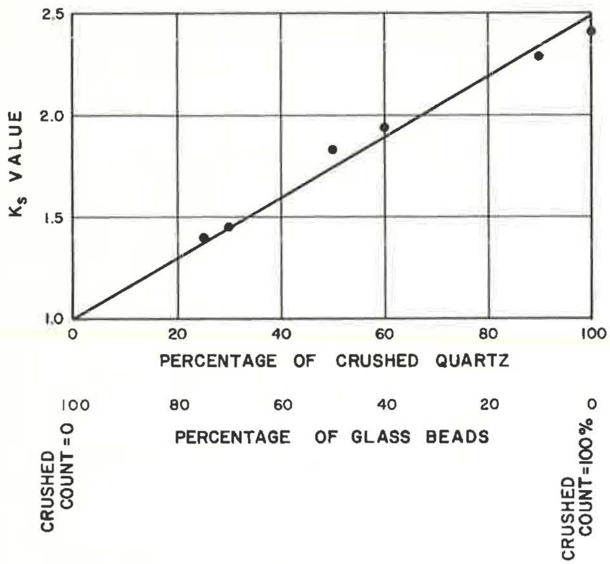
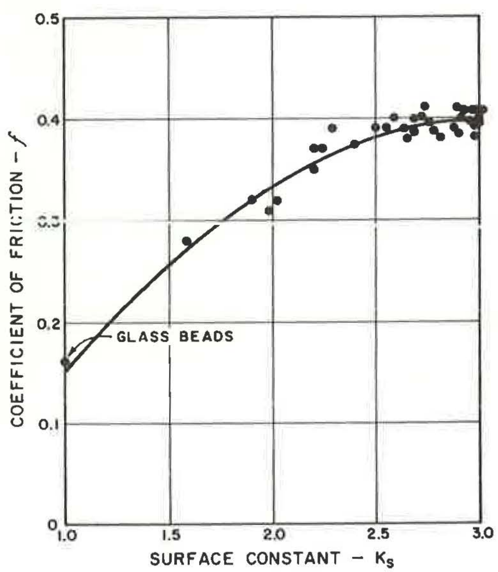


Figure 3. Change in K_s value on addition of crushed quartz to glass beads.



COEFFICIENT OF FRICTION VALUES DETERMINED AT 50 MI/HR. WITH WET PAVEMENT, SMOOTH TIRES AND LOCKED WHEELS. TESTS PERFORMED ON LABORATORY PREPARED TEST PLATES.

Figure 4. Relation between surface constant K_s and coefficient of friction of laboratory prepared test plates.

TABLE 3

RELATION BETWEEN SURFACE CONSTANT-K_s AND COEFFICIENT OF FRICTION OF LABORATORY PREPARED TEST PLATES

Material	K_s Value	Coefficient of Friction f
Glass beads	1.00	0.16
Screenings	1.58	0.28
Screenings	1.90	0.32
Screenings	1.98	0.31
Screenings	2.02	0.32
Screenings	2.20	0.35
Screenings	2.20	0.37
Screenings	2.24	0.37
Screenings	2.28	0.39
Screenings	2.40	0.37
Screenings	2.50	0.39
Screenings	2.52	0.39
Screenings	2.58	0.40
Screenings	2.64	0.39
Screenings	2.66	0.38
Screenings	2.68	0.385
Screenings	2.70	0.40
Screenings	2.72	0.40
Screenings	2.74	0.41
Screenings	2.76	0.395
Screenings	2.78	0.385
Screenings	2.82	0.38
Screenings	2.88	0.39
Screenings	2.90	0.385
Screenings	2.90	0.41
Screenings	2.92	0.40
Screenings	2.92	0.405
Screenings	2.96	0.405
Screenings	2.96	0.395
Screenings	2.98	0.405
Screenings	2.98	0.38
Screenings	2.99	0.39
Screenings	2.99	0.39
Screenings	3.00	0.40
Screenings	3.00	0.395
Screenings	3.00	0.405

[a]Coefficient of friction values determined at 50 mph with wet pavement, smooth tires and locked wheels.

The K_s factor has been determined for a large number of screening sources in California, and shows a range from 1.1 to 3.0. Typical screenings having varying K_s values are shown in Figure 2. Glass beads, 100 percent passing the No. 3 and all retained on a No. 4 sieve (B.K.H. size No. 6) have a K_s value of 1.0. This should be an absolute minimum K_s value since the glass beads are spherical and have no surface roughness. When increasing amounts of crushed quartz are added to glass beads, the K_s value of the combination increases in a linear manner (Table 2 and Fig. 3). The results clearly indicate that K_s values are a measure of angularity and surface roughness. The K_s value also may be used in place of the presently used qualitative crush count method for screenings.

The fact that the K_s value provides a measure of angularity and surface roughness indicates that the value should also be related to the skid resistance of the screenings.

This was confirmed by testing screening test plates with our skid tester and comparing these values with the K_s factors for various screening sources. The results are shown in Table 3 and Figure 4. The correlation obtained is considered excellent and indicates that the K_s value may be used to control the original skid resistance value of seal coat screenings.

TEST FOR WEAR AND POLISH

A most comprehensive study concerning the wear and polish characteristics of screenings has been conducted by the British Road Research Laboratory (1). They found a satisfactory correlation when the friction factors of laboratory polished screenings were compared with those obtained from test patches placed on the roadway. The results of this study indicated that methods could be developed for predicting future wear and polish of screening sources, and thereby prevent the use of screenings that would have a rapid reduction in coefficient of friction during traffic action. Therefore, an apparatus was constructed which permitted the polishing of our standard laboratory prepared plates. The apparatus (Fig. 5) consists of two wheels with 8.00 × 16.00 tires mounted on a revolving unit. The wheels move sideways over the seal coat test plate as the assembly revolves. This permits tracking over the entire test plate area. The assembly rotates at a speed of 13 rpm with a minimum radius of 24 in. and a maximum of 38 in. The speed of the wheel varies from 2.7 to 3.4 mph, depending on the radius. The movement along the shaft is actuated by a screw-type cut in the shaft and a key which kicks out when the wheel reaches the inner end of its travel. By providing the wheels with a slight toe out, they automatically return to the starting point where the key is released and caused to mesh with the threads of the screw.

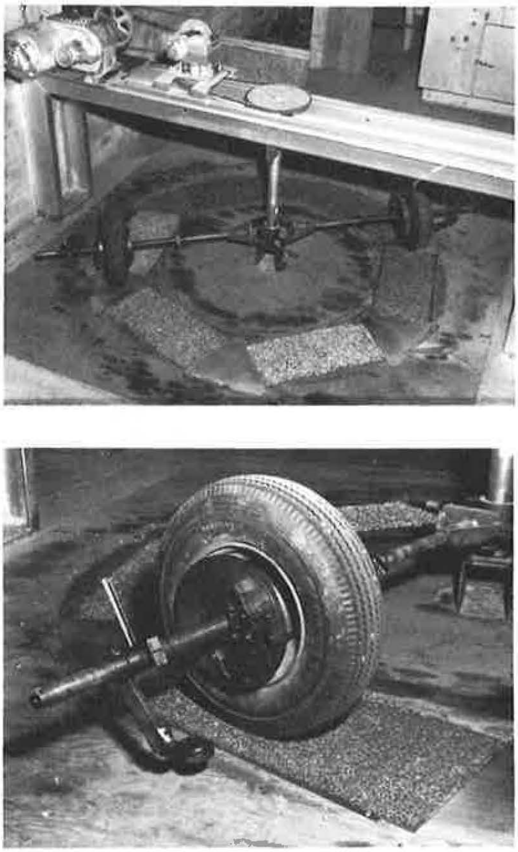


Figure 5. Laboratory polishing machine.

The seal coat test plates are anchored by triangular sheets of galvanized iron attached to the plywood floor with wood screws (Fig. 5). Preliminary studies indicated that some form of temperature control was required, since high atmospheric temperatures permitted excessive movement of the screenings during the circular movement of the wheels. Therefore, the entire assembly was enclosed in an air-conditioned room and the temperature maintained at 80 ± 5 F.

Screenings were obtained from various commercial sources in California. Most of the samples were medium-fine, $^5\!/_{16}$ in. × No. 8, the most commonly used size for seal coat work. Twenty-one samples representing various sources were chosen for the wear and polish study. Test plates were placed in the laboratory polishing unit, and periodically removed for skid resistance measurements. Each plate was subjected to a total of one million passes. No abrasive materials were used during the test. Friction values before and at intervals through one million passes are given in Table 4; typical wear and polish curves are shown in Figure 6. All screening samples showed a rapid drop in friction values during the first 200, 000 or 300, 000 passes, and thereafter attained an equilibrium figure. There was no further evidence of wear and polish up to one million passes.

TABLE 4

CHANGE IN FRICTION VALUES DURING SIMULATED TRAFFIC ACTION BY LABORATORY POLISHING MACHINE

Code No.	× 10^3 Passes						
	Orig.	15	60	160	355	670	1000
1	0.42	—	—	—	0.35	0.32	0.34
2	0.43	0.41	0.42	0.36	0.32	0.31	0.32
3	0.41	0.42	0.36	0.32	0.34	0.31	0.33
4	0.35	0.34	0.35	0.33	0.31	0.30	0.33
5	0.38	0.36	0.37	0.34	0.31	0.29	0.31
6	0.42	0.41	0.40	0.35	0.35	0.30	0.34
7	0.40	0.43	0.38	0.35	0.33	0.32	0.35
8	0.42	0.42	0.43	0.36	0.34	0.33	0.33
9	0.41	0.38	0.38	0.36	0.34	0.31	0.33
10	0.38	0.41	0.36	0.34	0.34	0.29	0.33
11	0.40	0.40	0.34	0.31	0.32	0.30	0.32
12	0.40	0.41	0.41	0.35	0.31	0.31	0.33
13	0.39	0.41	0.37	0.33	0.32	0.30	0.33
14	0.41	0.43	0.44	0.39	0.36	0.33	0.37
15	0.43	0.42	0.38	0.33	0.33	0.31	0.34
16	0.40	0.41	0.42	0.37	0.35	0.31	—
17	0.40	0.42	0.43	0.37	0.33	0.33	0.35
18	0.41	0.40	0.41	0.34	0.33	0.30	0.32
19	0.41	0.43	0.40	0.35	0.34	0.31	0.32
20	0.37	0.37	0.35	0.32	0.30	0.29	0.31
21	0.41	0.41	0.37	0.34	0.31	0.30	0.32
Avg.	0.40	0.40	0.39	0.35	0.33	0.31	0.33

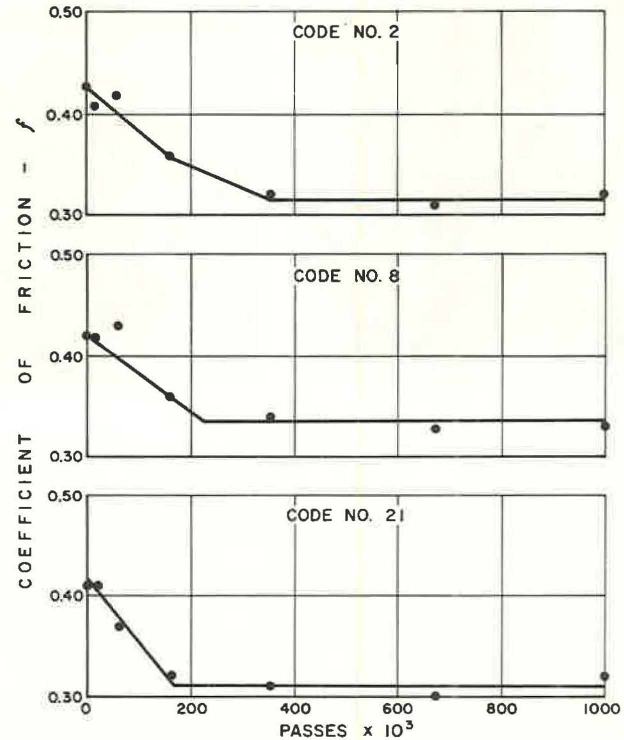


Figure 6. Change in friction values during laboratory simulated traffic action.

APPLYING EMULSION

SPREADING AND INITIAL ROLLING OF SCREENINGS

FINAL COMPACTION WITH A LIGHT DUTY TRUCK

OVERALL VIEW OF AUBURN TEST SECTION AFTER 6 DAYS OF TRAFFIC

Figure 7. Procedure for construction of field test sections.

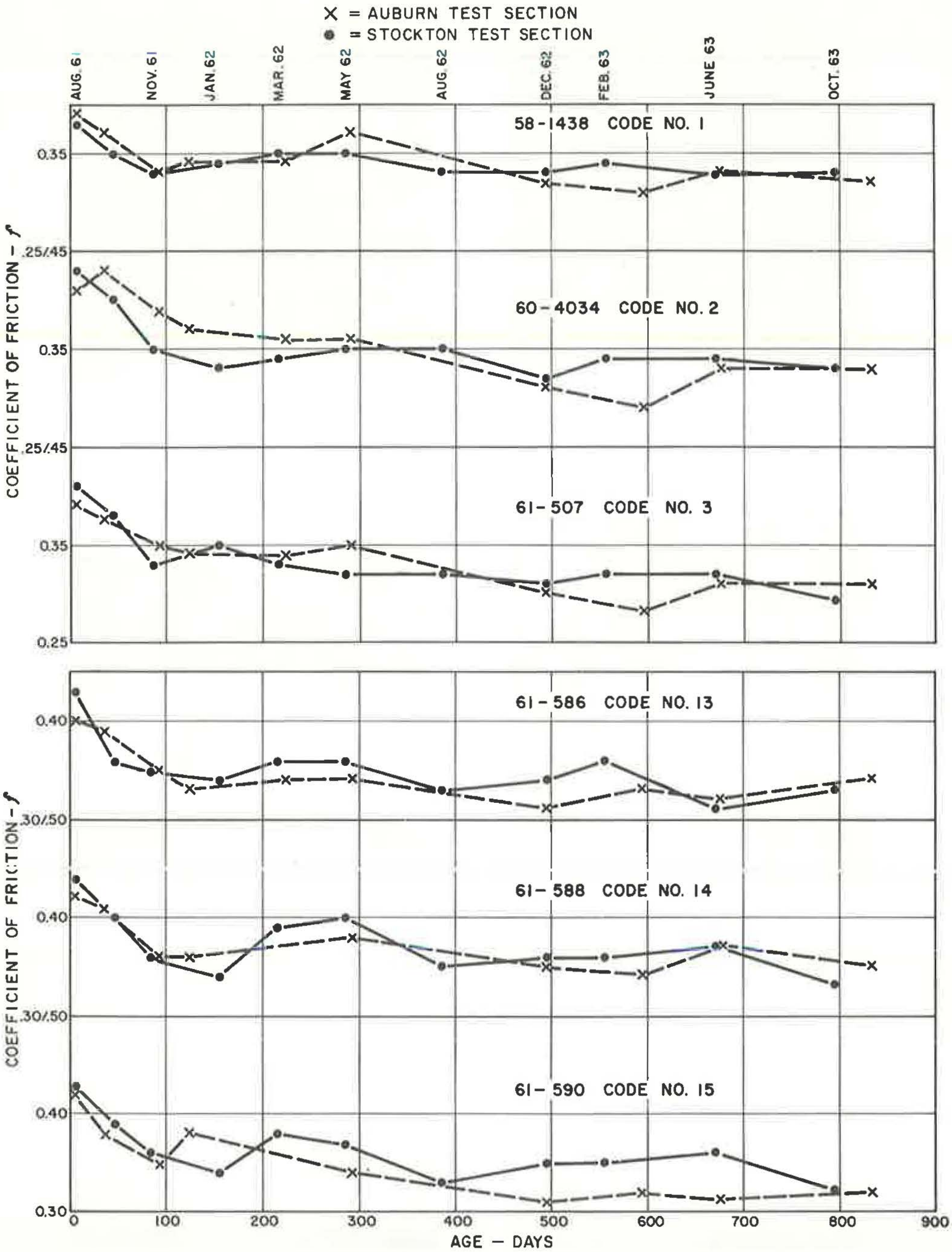


Figure 8. Change in friction values for various California seal coat screening sources under field traffic.

FIELD STUDIES

The results from laboratory polishing of screenings indicated that a state of equilibrium was attained in the coefficient of friction values after approximately 200 to 300 thousand passes. To verify this under actual traffic, it was decided to place test patches on two heavily traveled roads, and determine if a correlation existed between the equilibrium results obtained from the laboratory polishing unit and those obtained under traffic.

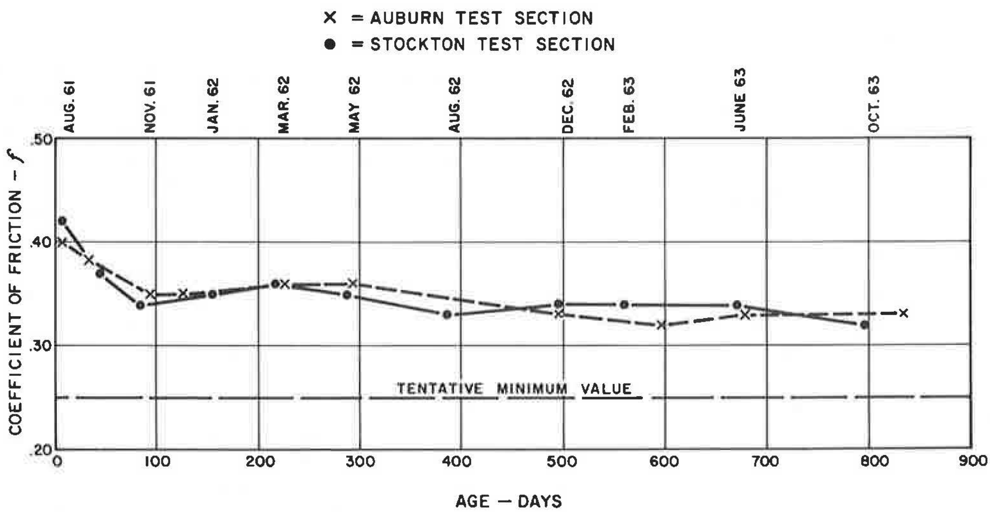


Figure 9. Change in average friction values for various California seal coat screening sources under field traffic.

TABLE 5

CORRELATION OF LABORATORY SCREENING POLISHING UNIT RESULTS WITH FIELD TEST SECTIONS AFTER TRAFFIC

Code No.	Equilibrium (f) Lab. Test Plates	Latest f-Value of Field Test Sections		Avg.
		Auburn 831 Days	Stockton 796 Days	
1	0.34	0.32	0.33	0.325
2	0.32	0.33	0.33	0.33
3	0.33	0.31	0.29	0.30
4	0.32	0.31	0.30	0.305
5	0.30	0.31	0.32	0.315
6	0.34	0.34	0.35	0.345
7	0.35	0.34	0.35	0.34
8	0.33	0.34	0.36	0.35
9	0.33	0.31	0.33	0.32
10	0.33	0.31	0.33	0.32
11	0.31	0.28	0.32	0.30
12	0.32	0.33	0.29	0.31
13	0.32	0.34	0.33	0.335
14	0.35	0.35	0.33	0.34
15	0.33	0.32	0.32	0.32
16	0.33	0.34	0.36	0.35
17	0.34	0.36	0.34	0.35
18	0.32	0.34	0.30	0.32
19	0.32	0.34	0.33	0.335
20	0.31	0.33	0.28	0.305
21	0.31	0.35	0.29	0.32
Avg.	0.33	0.33	0.32	

Two locations were chosen for field testing. The first is on I-80 about 2 miles west of Auburn, and the second on Calif. 99, between Stockton and Manteca. Both roads are 4-lane freeways and carry over 12,000 vpd with about 12 percent consisting of trucks.

Each test patch, 2 by 4 ft in area, was placed in the outer wheeltrack of the travel lane. The sequence of patch preparation is shown in Figure 7. The measured amount of emulsion, 0.14 gal/sq yd to 0.25 gal/sq yd, depending on screening size, was poured

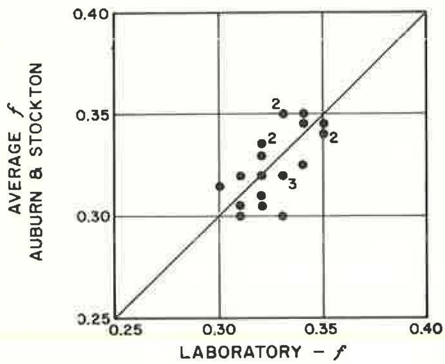


Figure 10. Correlation of laboratory screening polishing unit results with field test sections after traffic.

on the surface of the existing pavement and spread uniformly by a squeegee. Chips were then spread with a square-point shovel which produced a very uniform spread. The completed seal coat patch was then rolled with a hand roller. To avoid contamination, every other patch was placed, and by the time that these were completed, the first had set up sufficiently so that they could be walked on and the edges swept clean without damage. After completion of all the patches, they were rolled for approximately 2 hours with a light duty truck, and cured for approximately 4 hours before opening to traffic. The initial coefficient of friction readings were attained after 7 days of traffic. This time delay after placement allowed whip-off to be completed. Since completion in August 1961, periodical measurements of the coefficient of friction have been performed. Typical curves are shown in Figure 8. The greatest decrease in coefficient of friction occurs during the first six months. Thereafter, there is little decrease up to 800 ± days and it is apparent that equilibrium conditions have substantially occurred for all 21 screening sources. This is the same type of curve found for the laboratory polishing unit. Further, Maclean and Shergold (1) state: "The stones on a straight length of road approached their ultimate (equilibrium reading) in three to four months." Our studies, therefore, are in excellent agreement with those found by the British Road Research Laboratory. The average curves for all screening sources at both test sections are shown in Figure 9. There are indications, at least for the Stockton test section, that the winter rains improve the coefficient of friction over that found during the summer. Maclean and Shergold also found this in their studies. They state: "When a period of wet road surface conditions preceded the testing of the areas of stone chippings the 'skid resistance' values were increased. This effect was found to be associated with an actual roughening of the surface of the stones. On further investigation, it was found that the detritus on the road surface was coarser during wet than during dry conditions. As was established by laboratory investigations, the presence of coarser detritus would result in a roughening of the stones. It is thus apparent that polishing of stones is facilitated during summer when the road surface is predominantly dry, and delayed or reversed in winter when the surface is predominantly wet."

LABORATORY POLISHING UNIT: FIELD TEST SECTION CORRELATION

As previously mentioned, the primary purpose of the field test sections was to provide information for a possible correlation with the laboratory polishing unit. Results are shown in Table 5. Since the final friction values at Stockton and Auburn are almost the same, an average of the values is shown in Figure 10. Although screenings were obtained from various California sources, unfortunately all of the screenings have good resistance to wear and polish, and the equilibrium values tend to form a cluster within a range of 0.30 - 0.35f. The correlation appears quite satisfactory, considering the normal variations in the skid resistance test, and the fact that no screening source showed a high degree of wear and polish.

The equilibrium friction values obtained after subjecting a screening test plate to the laboratory polishing machine appears to check very closely with the equilibrium value attained under heavy traffic action. This test method may, therefore, be used for selecting screenings that will show a minimum reduction in coefficient of friction values due to wear and polish under traffic.

CONCLUSIONS

1. The original friction values on laboratory prepared screening test plates correlated with field test panels that had been subjected to 7 days of traffic in order to remove excess screenings. An excellent correlation was found between the K_S value determined by laboratory tests and the original friction value of laboratory prepared test plates. Therefore, it is possible to approximate the original coefficient of friction of screenings with a simple test method.

2. A satisfactory correlation between a laboratory wear and polish apparatus and wear and polish under traffic has been established. Therefore, the wear and polish characteristics of screenings may be specified, if necessary, by a minimum friction value after a specified period of simulated traffic action in a laboratory apparatus.

3. There appears to be a slight increase in the coefficient of friction of the field test patches after winter rains.

4. Screenings from various California sources do not show excessive reduction in friction values under heavy traffic, and appear to have low rates of wear and polish.

RECOMMENDATIONS

Our studies indicate that it is possible to predetermine the original skid-resistance characteristics of seal coat screenings by a simple test that provides a quantitative measure of the angularity and surface roughness. We have set a minimum value for K_S of 2.2 which will insure a minimum friction value of 0.35f. The use of this test will insure uniform characteristics for screenings together with a high initial friction factor.

The determination and control of the original friction value will not insure against a rapid change in friction characteristics by the effect of traffic wear and polish. When tested in a unit of the type described, the friction value at equilibrium should not be lower than 0.25f. We believe that commercial plants which produce screenings from a constant aggregate source would only require an occasional wear and polish test. It appears that control of shipments could be maintained by use of the K_S value test.

ACKNOWLEDGMENT

This paper is based in part on data collected during a research project financed jointly by the California Division of Highways and the U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads.

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Appendix A

MATERIALS AND RESEARCH DEPARTMENT

Test Method No. Calif. 342-C

April, 1966

METHOD OF TEST FOR PAVEMENT SURFACE SKID RESISTANCE

SCOPE

This method describes the apparatus and procedure for obtaining surface skid resistance values of Bituminous and Portland Cement Concrete pavements.

PROCEDURE

A. Apparatus

1. Skid test unit.

a. Reference is made to Figures I through III in connection with the following description of the construction of the test unit. A 4.80/4.00 × 8, 2-ply tire with (25 ± 2 psi) air pressure (A), manufactured with a smooth surface, together with rim, axle, and driving pulley is mounted on a carriage (B). The tire is brought to desired speed by motor (H). The carriage moves on two parallel guides (C), and the friction is reduced to a low uniform value by allowing three roller bearings fitted at 120° points to bear against the guide rod at each corner of the carriage. The bearing assembly may be noted on Figure III (D). The two guide rods (C) are rigidly connected to the end frame bars (E). The front end of this guide bar frame assembly is firmly fastened to a restraining anchor. The bumper hitch provides for swinging the skid tester to the right or left after positioning the vehicle. The rear end of the frame assembly is raised by a special adjustable device (F), Figure II, so as to hold the tire ¼-inch off the surface to be tested. This device is so constructed that the tire may be dropped instantaneously to the test surface by tripping the release arm (G), Figure II. Tachometer (K) indicates the speed of the tire.

2. Hitch for fastening unit to vehicle.

3. Special level to determine grade of pavement.

a. A 28" long standard metal carpenter's level, Fig. IV, is fitted at one end with a movable gauge rod which is calibrated in % of grade.

B. Materials

1. Glycerine
2. Water
3. 2-inch paint brush
4. Thickness gauge ¼-inch (a piece of ¼-inch plywood 2' × 1" is satisfactory).

C. Test Procedure

1. Determine and record grade with special level, see Fig. IV.

a. Place level on pavement parallel to direction of travel with adjustable end down grade.

b. Loosen locking screw and raise level until bubble centers and then tighten locking screw on sliding bar.

c. The grade is indicated on the calibrated sliding bar.

2. Remove apparatus from vehicle and attach to bumper hitch, Fig. V.

3. Position apparatus with tire over selected test area and parallel to direction of traffic.

4. Raise tire and adjust to ¼-inch (—1/16" tolerance) above surface to be tested with device (F).

5. Wet full circumference of tire and pavement surface under tire and 16" ahead of tire center with glycerine, using a paint brush.

6. Set sliding gauge indicator (P) against carriage end.

7. Depress starting switch (J) and bring the speed to approximately 55 mi/hr.
8. Release starting switch.
9. The instant the tachometer shows 50 mi/hr. trip arm (G) dropping tire to pavement.
10. Read gauge (N) and record.
11. Release rebound shock absorber.
12. Move to next section and repeat.
13. In any one test location, test at 25' intervals in a longitudinal direction over a 100' section of pavement.

D. Precautions

1. The rear support rod (O), Fig. II, must be cleaned by washing frequently with water and a detergent to prevent sticking.
2. Sliding gauge indicator (P) must be kept clean so that it will slide very freely.
3. On slick pavements glycerine remaining on the pavement should be flushed off with water to prevent possible traffic accidents.

E. Field Construction Testing of Portland Cement Concrete Pavement

The following procedure shall be followed in the field testing of a portland cement concrete pavement for specification compliance of the minimum friction value. A minimum of seven days after paving shall lapse before testing.

1. The total length of pavement shall be visually surveyed for uniformity of surface texture. All areas which do not have definite striations or which appear smooth shall be noted. This survey should be conducted with the Resident Engineer or an Assistant who has knowledge of any difficulties in attaining a proper surface texture during construction. The attached photograph, Figure VIII, may be used as an aid in the evaluation of the existing texture in relation to the coefficient of friction, but is not to be used in lieu of actual coefficient of friction measurements.

2. The determination of test locations, as outlined below, shall apply only to that portion of the pavement which has well formed striations. All areas that appear smooth, or those that have been ground shall be excluded.

a. A minimum of three test locations shall be made for each days pour and a minimum of three pour days shall be checked per contract.

The location of test sites shall be determined in a random manner through use of a Random Number table. The use of this method requires that the area for test be uniformly textured and placed in one operation. As an example, a 4-lane pavement may be placed with a three lane width in one operation and the fourth lane placed separately. Each of these areas must be treated separately in selecting test locations. The following example illustrates the use of this table.

A section of pavement is 24' wide and 4000' long and is part of a 4-lane freeway. This section of pavement has been placed in one operation and skid tests are required. From 2-a, it is required that three test locations be determined.

Using the random numbers, as shown, choose the three locations in the following manner:

Random Numbers

Longitudinal	Lateral
0.6	6
0.9	9
0.2	2
0.7	7
0.5	5
0.1	11
0.4	4
0.8	8
0.3	3

Starting at any point and proceeding up, or down, but not skipping any numbers, read three pairs of numbers and set up each location as follows:

	Distance from Start of Pour	Distance from Right Edge of Pour Looking up Station
Location A	0.6 × 4,000' = 2,400'	6 × 2 = 12'
Location B	0.9 × 4,000' = 3,600'	9 × 2 = 18'
Location C	0.2 × 4,000' = 800'	2 × 2 = 4'

In case any location as determined above falls in a smooth or ground area which does not appear representative of the general surface texture, then the next number in the random table shall be chosen and a new location selected.

At each test location the first reading shall be obtained at the specified random location. The next four readings shall be obtained at 25' intervals beyond the first reading. All readings shall be obtained at sites parallel to the centerline of the lane. After correction for grade as shown in F, average the five readings. This average shall be recorded as the friction value for the specific test location.

3. In all areas that present a smooth textured appearance or have been ground, the following shall apply:

a. A minimum of three ground area locations and all smooth appearing surfaces shall be checked on each contract.

b. If the area is less than 100' in length perform at least three individual tests in separate spots, correct for grade and average the results.

c. If the area is greater than 100' in length, sufficient test locations shall be selected to insure that the area is above the minimum requirement. If the average value of all locations is below the required minimum then additional tests shall be performed until the area is localized for remedial action.

F. Calculations

1. Make grade corrections using charts shown in Figures VI and VII.

2. Average the 5 corrected readings in any one test location. Example—The following readings were taken at 25' intervals in a test location. The grade of the pavement, determined as described in C-1, was +4%.

Station	Measured Coefficient of Friction	Corrected Coefficient of Friction*
1+00	0.33	0.38
1+25	0.34	0.39
1+50	0.34	0.39
1+75	0.33	0.38
2+00	0.33	0.38

Final Average for Test Site. 0.38

*Corrected coefficients of friction were taken from chart in Figure VI.

G. Reporting of Results

For all results determined under E-2, report the result for each station location and the average of 5 readings and the grand average. For all results determined under E-3, part (b), report the result for each station location and the average. For E-3, part (c), report the result for each station location and the average for each set of five determinations.

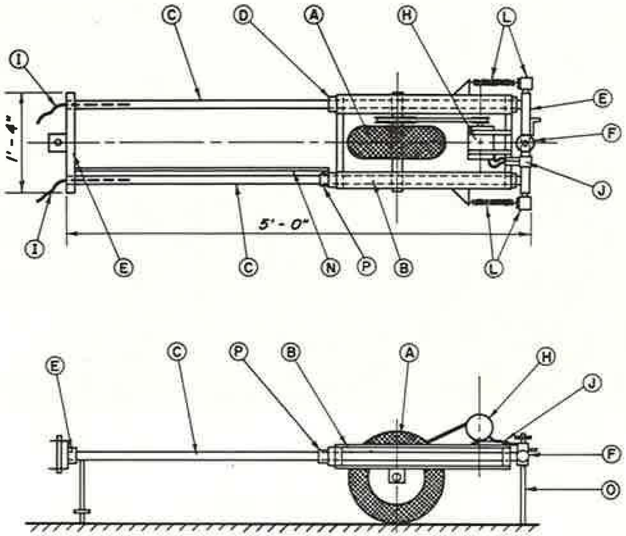

DIAGRAM OF SKID TESTER

Figure I

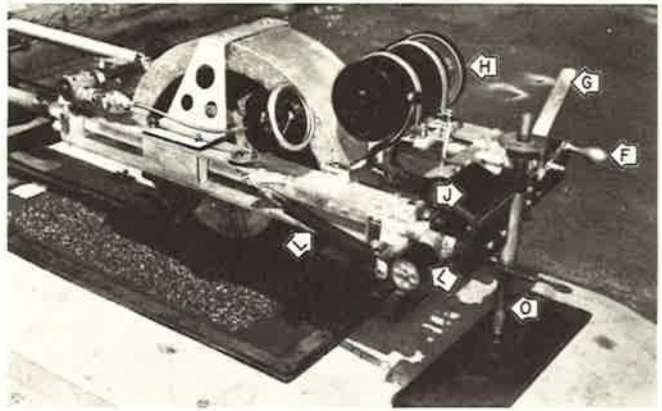

Figure II

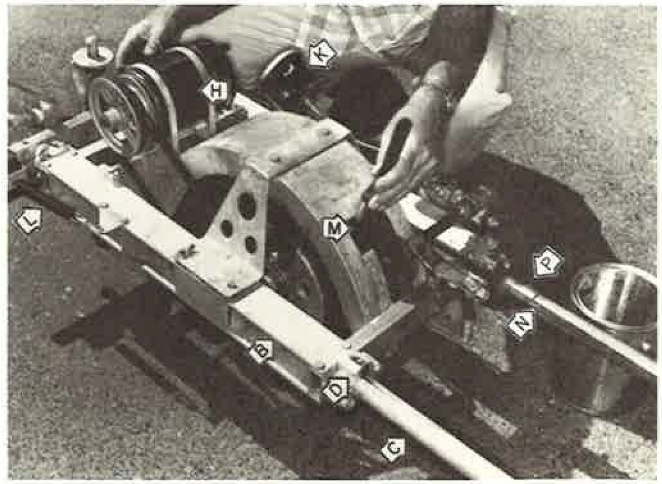

Figure III

Close-up Views of Skid Tester

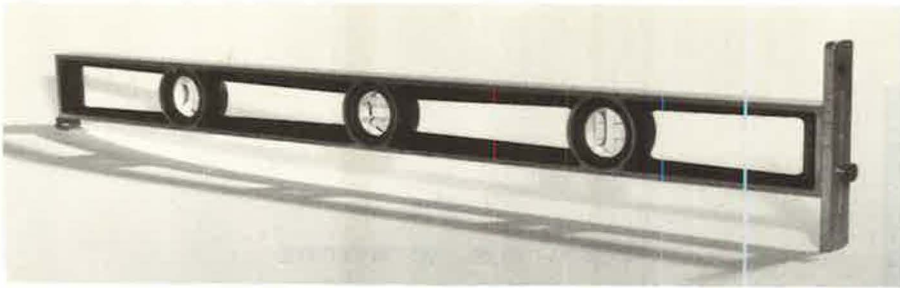

Figure IV

Level for Determining Grade

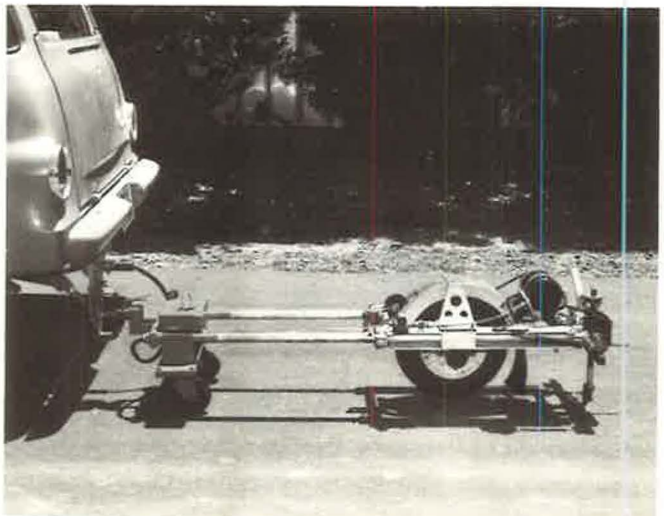

Figure V

Apparatus in Position for Testing

COEFFICIENT OF FRICTION CORRECTION CHART
FOR MEASUREMENTS MADE ON GRADES

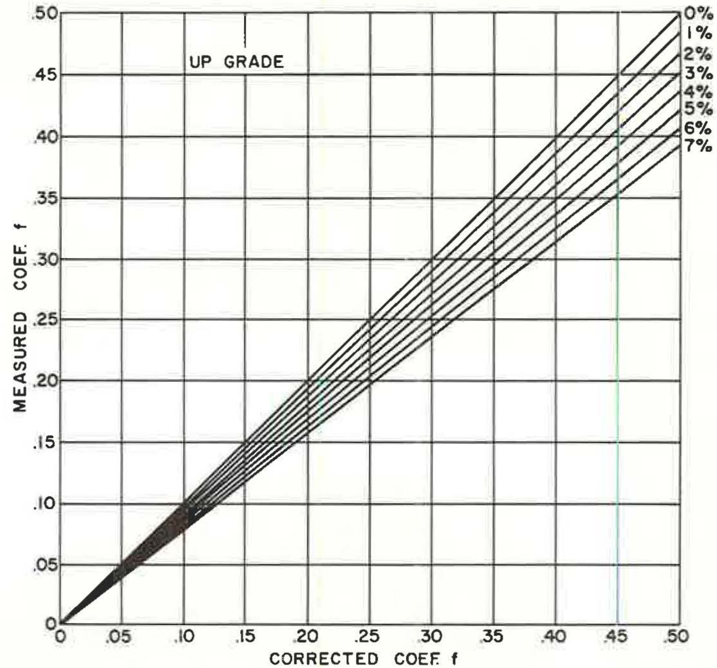

Figure VI

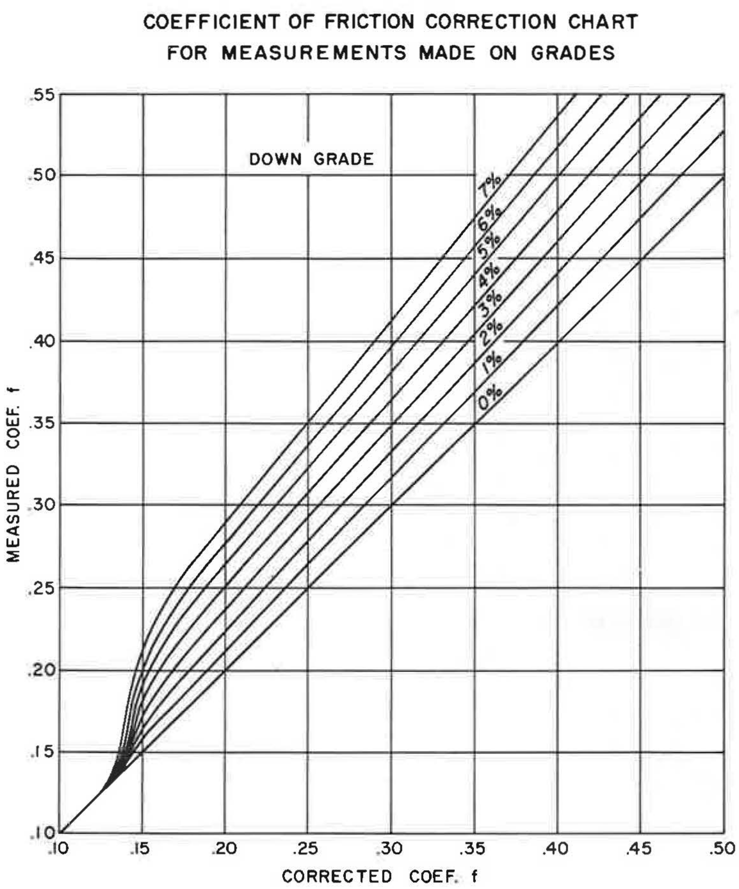

Figure VII

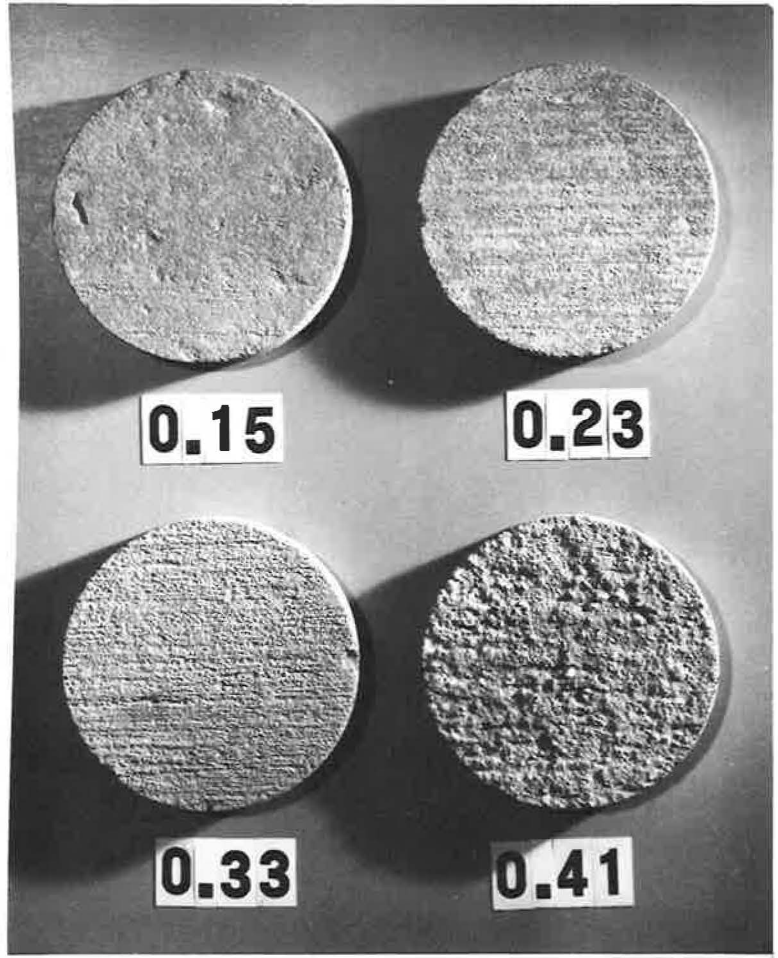

Figure VIII

Photos of Surface Textures

Figure IX

Apparatus Being Placed in Vehicle
Note cable and winch for moving skid tester.

Figure X

Apparatus in Position for Transportation

Appendix B

METHOD OF TEST FOR ANGULARITY AND SURFACE ROUGHNESS OF SCREENINGS

SCOPE

The test furnishes a measure of the surface capacity as it is affected by surface texture and particle shape characteristics.

PROCEDURE

A. Apparatus

1. Centrifuge (hand or power driven) capable of exerting a force of 400 times gravity (400G) on a 100 gram sample

$$\text{Required RPM of centrifuge head} = \sqrt{\frac{14,000,000}{r}}$$

Where r = radius in inches to center of gravity of sample.

2. Centrifuge cups $2^{13}/_{16}$" in height and $2^{1}/_{16}$" in diameter complete with perforated brass plate, .031" thick with a minimum of 100 holes, .062" in diameter, per square inch.
3. Torsion balance, 500 g. capacity ± 0.1 g. accuracy.
4. Metal funnels, top diameter $3^{1}/_{2}$", height $4^{1}/_{2}$", orifice $^{1}/_{2}$", with a piece of No. 10 sieve soldered to the bottom of the opening.
5. Glass beakers (1, 500 ml.).
6. Timer with sweep second hand.
7. 140°F oven.
8. Hot plate or 230°F oven
9. Small drain pans approximately 5" in diameter by 1" deep.

B. Materials

1. Kerosene.
2. S.A.E. No. 10 lubricating oil.
3. Filter paper, Eaton-Dikeman Co. size $5^{1}/_{2}$ cm. No. 611.

C. Test Record Form

Use work form T-302 for recording test data.

D. Preparation of test sample

Screen sufficient material to obtain approximately 100 g. of aggregate passing a No. 3 sieve and retained on a No. 4 sieve. Wash this sample clean, dry thoroughly and allow to cool.

E. Tests and calculations

1. Nomenclature
   a. K_S = a factor arrived at by subtracting the weight of the test sample after soaking in kerosene and centrifuging from the weight after submerging in the lubricating oil and draining.
   b. K_S corrected - the factor K_S corrected for specific gravity of the aggregate.
2. Procedure for K_S
   a. Weight to the nearest 0.1 gms. 100 gms. ± 1.0 gm. of the washed and dried test sample.
   b. Place sample in tared centrifuge cup and submerge sample and cup in a beaker of kerosene for 10 minutes.
   c. Remove sample and cup from kerosene and centrifuge for 2 minutes at 400 times gravity (400G).
   d. Remove from centrifuge and re-weigh to the nearest 0.1 gm. Subtract original weight and record the difference as grams of kerosene retained.
   e. Immediately after obtaining the weight of kerosene retained, remove the sample from the cup and place in a funnel (standard C.K.E. test funnel) and submerge sample and funnel in a beaker of S.A. E. No. 10 lubricating oil, raise immediately and allow to drain for 2 minutes.
   f. Sample and funnel are then placed in a 140°F oven for 15 minutes additional draining time.

g. Remove from oven and empty sample from funnel into a small pan 4 or 5 inches in diameter and allow to drain for 1 minute at room temperature.
h. Empty sample from drain pan into a tared container and weigh to the nearest 0.1 gm.
i. Subtract from the final weight, the weight obtained after centrifuging, and record the difference which will be designated as K_S, representing a surface constant for the particular aggregate tested.
j. If the specific gravity of the aggregate is greater than 2.70 less than 2.60 make correction to K_S value as follows:

$$K_S \times \frac{\text{Sp. Gr. of Agg.}}{2.65} = K_S \text{ corrected}$$

k. Perform the test in triplicate and average the results.

F. Reporting of Results

Report the K_S value to nearest 0.1 gm. on Test Report Form T-374.

Skid Resistance of Bituminous Surfaces

W. C. BURNETT, J. L. GIBSON, and E. J. KEARNEY, Bureau of Physical Research, New York State Department of Transportation

Seven years of New York State research in skid-resistance measurement of bituminous surfaces are summarized. Development of a skid trailer and test procedure are discussed. Skid testing is recommended only after 5 million vehicles have passed over a surface, so that friction coefficient has declined to a stable level. Testing shows that largest mix aggregates, rather than fines, control skid resistance, which remains at a higher level when hard particles resistant to polishing are included.

•SINCE the early 1930's, highway engineers have been concerned with the problem of pavement slipperiness (1). Increasing volume and speed of motor vehicle traffic have made the skidding problem increasingly serious. In 1960, shortly after organization of the New York State Department of Transportation's Bureau of Physical Research, an investigation was undertaken to determine the factors controlling skid resistance of highway pavements.

First studies concentrated on bituminous pavements, which make up a large percentage of highway surfaces in New York State, both in initial construction and as overlays to upgrade pavements having low coefficients of friction. The study was limited to clean pavements, since even the most skid-resistant surface is slippery if covered by ice, snow, mud, or wet leaves. Pavements having excess asphalt on the surface were also excluded. The objectives of this research program were the following.

1. To select equipment and establish a procedure for measuring pavement coefficient of friction; and
2. To determine the influence of the following pavement factors on skid resistance: (a) mix type, (b) types of fine and coarse aggregate, (c) abrasion resistance of coarse aggregate, and (d) mineral composition of coarse aggregate.

MEASUREMENT SYSTEM AND PROCEDURE

Theoretically, coefficient of friction is independent of the contact area, speed, and weight of a sliding object. This is not true, however, when a rubber tire slides on a wet pavement. To obtain a true indication of pavement slipperiness, it is necessary either (a) to actually bring a car to a skidding stop on a wet pavement and measure its stopping distance, or (b) to build a device that closely reproduces the conditions of a skidding car and gives readings correlating with the stopping distance method.

The stopping distance method has several disadvantages. At least one lane of traffic must be closed and wetted; tests cannot be performed on grades, curves, or highly crowned roads; and accidents are always possible. Because of these limitations and because skid trailers can be correlated with the stopping distance method, New York State decided to use a drag-force-type trailer (Fig. 1) to measure the coefficient of friction of wet pavement, as described in a previous report (2).

In designing the trailer, a primary objective was to reproduce, as closely as possible, the geometry and mechanics of a skidding automobile. The New York State skid trailer has a wheel loading and suspension system typical of American passenger cars.

Paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction and presented at the 47th Annual Meeting.

Figure 1. Skid trailer and tow truck.

TABLE 1
MEASURING SYSTEM RELIABILITY

Pavement	Coefficient of Friction			
	Min.-Max.	Range	Std. Dev.	Avg.
x	0.53-0.58	0.05	0.015	0.55
y	0.40-0.42	0.02	0.007	0.41
z	0.33-0.36	0.03	0.010	0.35

The standard ASTM test tire (ASTM Designation E 249), used in all testing after 1961, represents a compromise between a bald tire and one designed to give high skid resistance.

Other skid research agencies had found that about 25 to 30 gpm of water should be applied to dry pavement for a one-wheel skid test. This amount is sufficient to prevent any portion of the tire from skidding on dry pavement. Water is applied to the pavement less than 1 ft in front of the tire, and a fairly large pipe opening and low water pressure minimize scour of road film. These test conditions were chosen as generally duplicating what happens on a pavement just as it begins to rain.

Skid resistance tends to vary with season. Lowest readings usually occur during summer and higher coefficients in winter. To minimize this variable, skid testing in this study was limited to summer months.

The coefficient of friction of wet pavements also varies with speed. It was desirable that one test speed be chosen for comparison of different pavements, preferably close to the travel speed of prevailing traffic. However, to test in suburban areas as well as open rural locations, 40 mph was chosen for this study. Testing at the speed limit (50 mph) would have been difficult and hazardous in some instances.

Tests were made on two pavements to determine variation of coefficient with speed. Pavement A with low skid resistance and Pavement B with a high coefficient were selected and tested at speeds ranging from 10 to 60 mph (Fig. 2). Friction decrease with increased speed differed on these two surfaces. Some property or characteristic of the pavement surfaces apparently caused the different rates of coefficient decrease. Further study is needed before coefficients measured for any given surface at one speed can be extrapolated to coefficients at another speed.

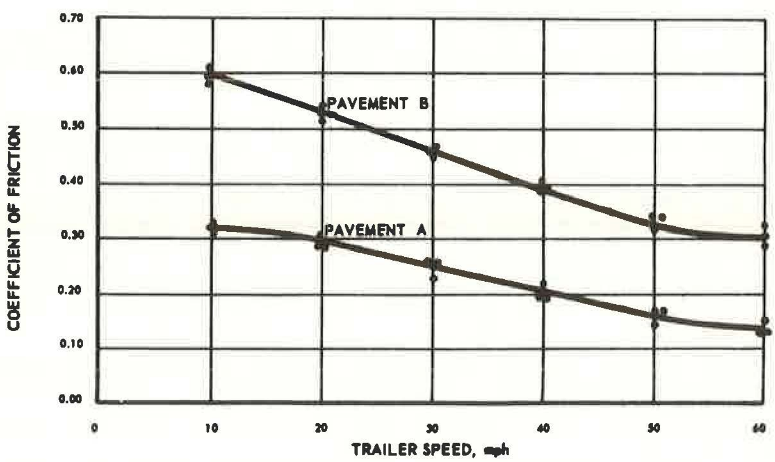

Figure 2. Effect of speed on coefficient of friction.

To determine the reliability of the measuring system, several tests were performed on sections of three different pavements. The resulting narrow range and small standard deviation (Table 1) indicate that the trailer yields highly reproducible measurements over a wide range of coefficients.

Trailer readings also correlate very well with distance required to stop a car. At the 1962 correlation study at Tappahannock, Va. (3), the New York trailer and Purdue University car each tested five surfaces having a wide range of friction coefficients. The car's stopping distance (measured from the point where brakes were applied, as identified by chalk fired at the pavement when wheels locked at 40 mph) was converted to a coefficient of friction (C_f) by the following:

$$C_f = \frac{V^2}{30S}$$

where

V = initial car speed, mph, and
S = stopping distance, ft.

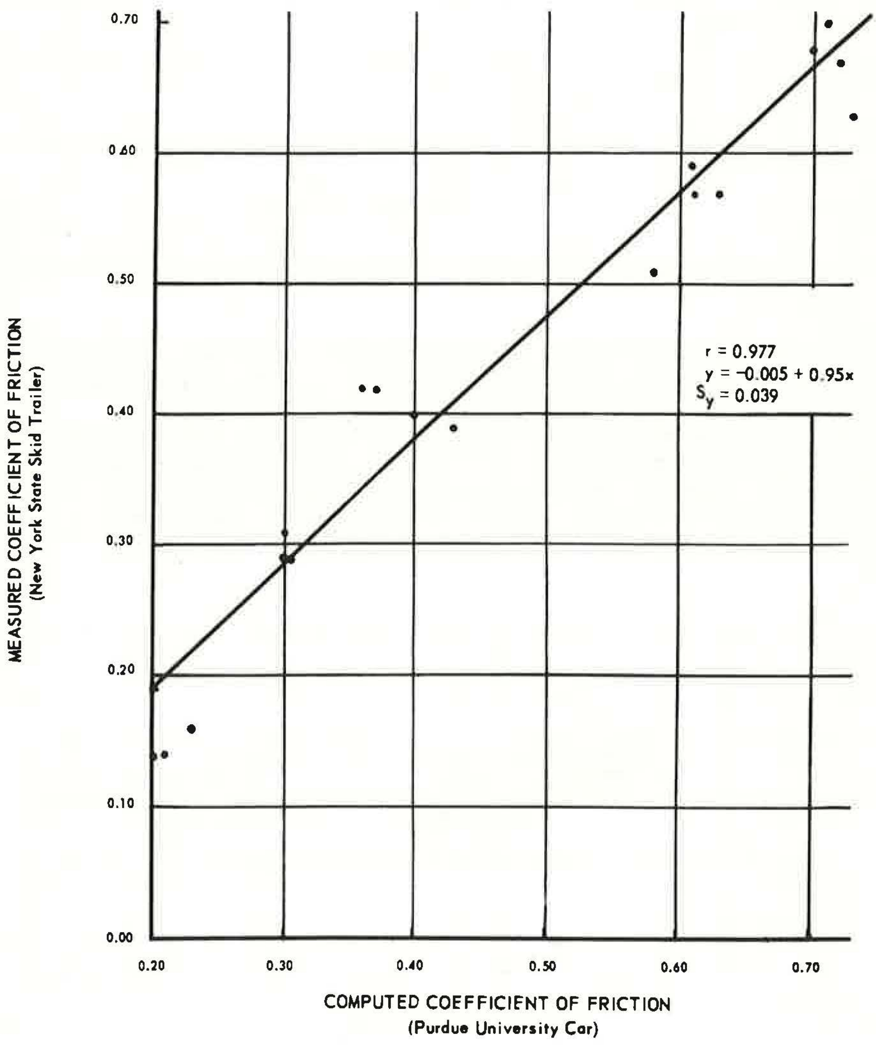

Figure 3. Correlation of two skid measuring methods.

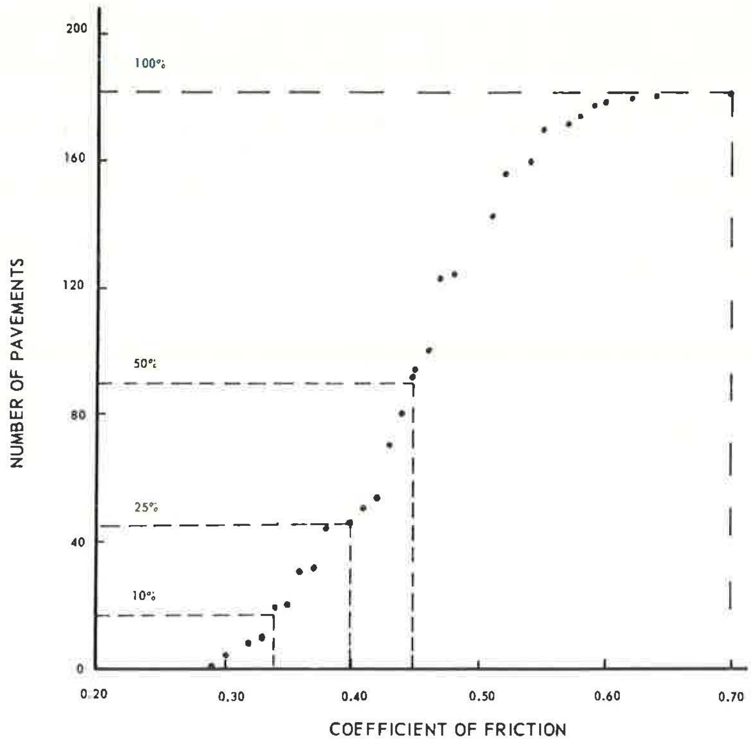

Figure 4. Cumulative frequency distribution of friction coefficient of new bituminous pavements.

Trailer and car coefficients were plotted in Figure 3. Trailer measurements were made at 40 mph using the truck's watering system. An excellent correlation was obtained [coefficient of correlation (r) = 0.977] between the trailer and car.

Initially, only new bituminous concrete pavements were skid tested because information concerning their construction material was readily available. Little was learned from this study, since recently placed pavements generally have high coefficients. Figure 4 is a cumulative frequency polygon showing results of skid tests on 182 bituminous pavements less than 3 months old; 75 percent had coefficients above 0.40.

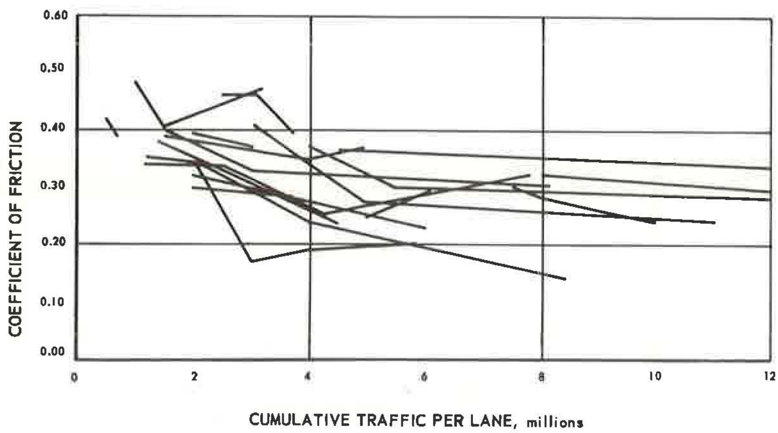

Figure 5. Effect of traffic on 19 limestone surfaces (with 1 truck equivalent to 11 cars).

Nineteen pavements tested during their first year were retested the following summer. In all cases, coefficients were lower. The effect of traffic on their skid resistance is indicated in Figure 5. In calculating traffic volume, each truck was assumed to have the same effect as 11 cars, an approximation made to take into account the greater number of tires, using harder rubber and higher inflation pressures. The lines in the figure connect the results of tests performed at different times on the same pavement. It was clear that as traffic used a pavement, skid resistance decreased, rapidly at first and then more gradually. After 5 million vehicles passed over a surface, a nearly stable coefficient was reached, dropping 0.02 or less with each additional 1 million vehicle passes. Therefore, it was decided that a pavement should be exposed to 5 million vehicle passes before final evaluation of its skid resistance.

In summary, to measure a pavement's coefficient of friction, the trailer is towed at 40 mph and water (25 gpm) is applied to the dry road directly in front of a standard test tire (ASTM Designation E 249). Either or both trailer wheels are locked for about 2 sec. Drag force is measured between the sliding tire and the wet pavement. This force divided by wheel load gives the pavement's coefficient of friction. Testing was limited to summer months and data evaluation to pavements that had carried more than 5 million vehicles.

LABORATORY STUDY

During the first 2 years of field testing, some pavements were found to have good skid resistance after 5 million vehicle passes, whereas others had low coefficients with the same traffic. A laboratory investigation was undertaken to determine which pavement variables, especially aggregate type, affected skid resistance.

A laboratory study was considered the fastest and most practical way to evaluate the many combinations of crushed stone and natural sand used in New York State top course mixes. An apparatus similar to that devised by Maclean and Shergold (4) simulated traffic wear of aggregates in a bituminous mix (Fig. 6). It consisted of a wheel with a bald pneumatic tire, driven by an electric motor. The tire rode on 20 wire-reinforced asphalt-concrete samples, which in turn were mounted on the perimeter of a 48-in. diam wheel revolving at 100 rpm. Dry aluminum oxide and carborundum grits were introduced between the tire and samples. The bald tire was chosen as providing uniform wear across the samples after it was found that a treaded tire left an undesirable pattern.

The British portable tester (BPT) was used to measure the skid resistance of the samples (Fig. 7). For its use, readings

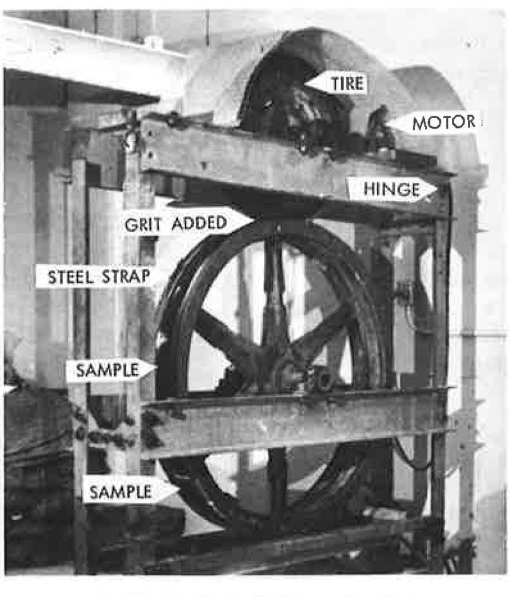

Figure 6. Polishing wheel.

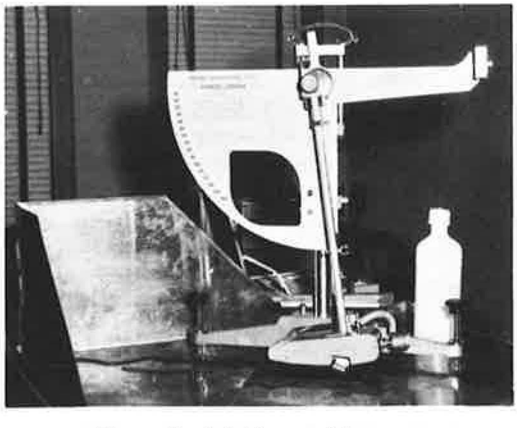
Figure 7. British portable tester.

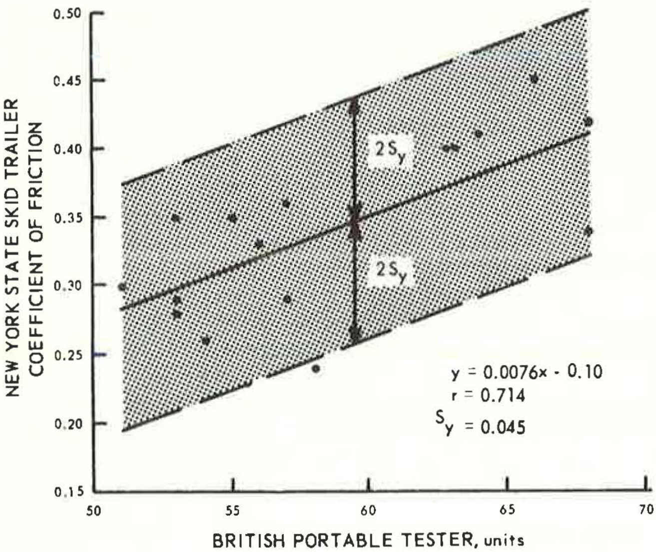

Figure 8. Correlation of British portable tester and New York State skid trailer (40 mph).

of the BPT and New York State trailer had to be correlated. Samples were taken from the wheelpaths of 16 pavements tested with the trailer. In Figure 8, skid trailer coefficients for these surfaces are plotted against BPT measurements taken on the samples at 78 F. Each point is an average of four or more readings with each machine. A fair correlation ($r = 0.714$) was obtained, but the standard error of estimate was large (0.045). A BPT value of 60 corresponds to a trailer coefficient of friction between 0.26 and 0.44. This confidence interval is too wide and covers almost the entire range of trailer coefficients for the samples.

However, since aggregates with known skid characteristics were included in the samples, the laboratory experiment was continued because it was thought that relative BPT values would be useful. Sieve size No. 80 grits were used to polish the samples for 240,000 revolutions of the sample wheel. BPT readings at that time ranged from 45 to 53 units for all surfaces, too narrow a range for evaluation of 20 different surfaces. For an additional 160,000 revolutions, grit size No. 240 was used as an abrasive. The range of BPT readings changed very little and no further polishing was attempted. Because of poor correlation between the trailer and portable tester, and the small range of BPT units on surfaces polished in the laboratory, this technique was abandoned.

INFLUENCE OF PAVING MIXES AND MATERIALS

During the 7 years the skid trailer has been in operation, tests have been performed on pavements constructed from nearly every aggregate source in the State. Measured coefficients of friction have been compared for many properties that vary in a bituminous mix.

Friction Coefficients of Common Mix Types

Three mixes are commonly used for New York State surface courses: asphalt concrete Types 1A and 1AC, and stone-filled sheet asphalt Type 2A. Mix specifications are summarized in Table 2. To determine whether mix types differed in skid resistance properties, tests were run on numerous surfaces of these types, all of which had carried over 5 million equivalent vehicles. Mean coefficients for the three mixes (Fig. 9) did not vary significantly at the 95 percent confidence level.

Effects of Fine Aggregates

Two studies were made to determine possible effects of fine aggregate on skid resistance. For the first, it was known that varying amounts of natural siliceous sand had been used in place of screenings as the fine aggregate in Types 1A and 1AC asphalt concrete mixes, in some Department of Transportation districts. This had been done on the assumption that the hard sand would help retain good skid resistance. Coefficients of friction of certain pavements (all with at least 5 million equivalent vehicle passes)

TABLE 2

BITUMINOUS PLANT MIXTURES FOR STANDARD TOP COURSES

Item	Type 1A General Limits (% passing)	Type 1A Job Mix (% tol.)	Type 1AC General Limits (% passing)	Type 1AC Job Mix (% passing)	Type 2A General Limits (% passing)	Type 2A Job Mix (% tol.)
Screen size:						
1 in.	100	±0	—	—	—	—
1/2 in.	95-100	±5	100	0	100	0
1/4 in.	65-85	±5	95-100	±5	90-100	±5
1/8 in.	32-65	±6	45-70	±6	65-80	±4
No. 20	15-39	±7	8-40	±7		—
No. 40	7-25	±6		—	35-70	±4
No. 80	30-12	±3	3-15	±3	17-40	±3
No. 200	2-6	±6	2-8	±2	5-12	±2
Asphalt cement:						
Percent	5.8-7.0	±0.4	6.0-8.0	±0.4	7.5-8.5	±0.4
Penetration	85-100		85-100		60-70	
Placing temperature	225-300 F		250-325 F		275-350 F	
Texture	Granular		Smooth-Gritty		Smooth-gritty	
Typical uses	Interurban, urban resurface, urban				Urban	

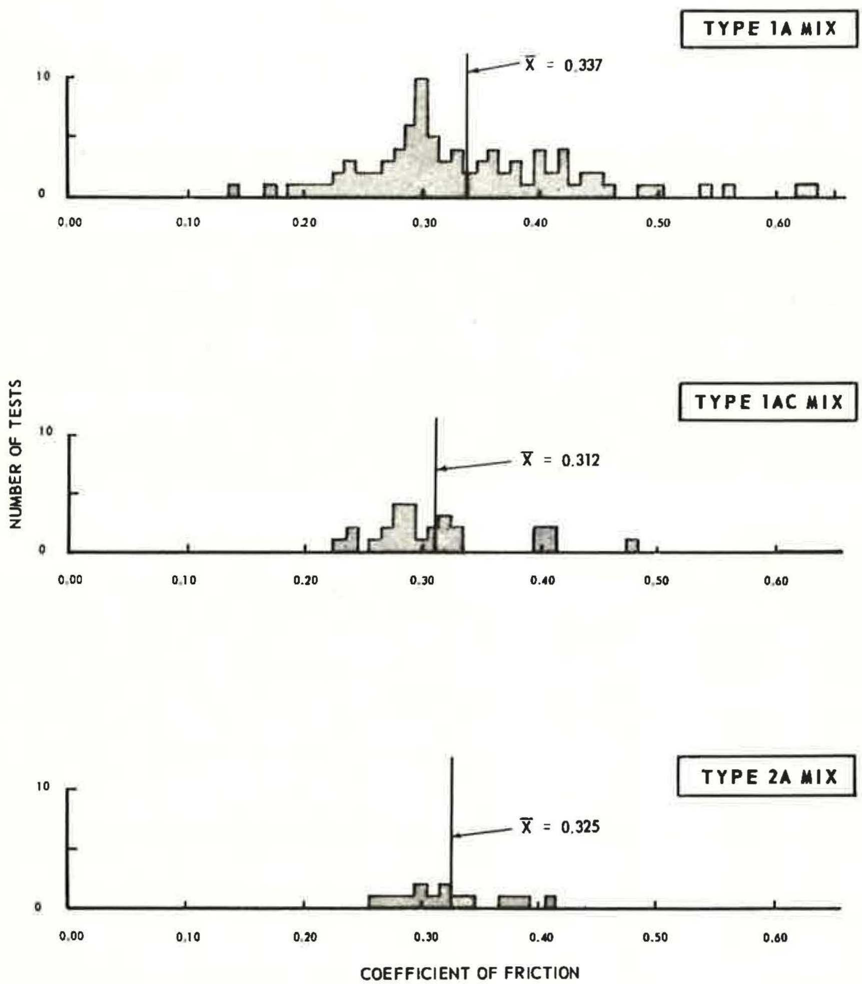

Figure 9. Skid resistance of three most common bituminous top courses specified by New York State.

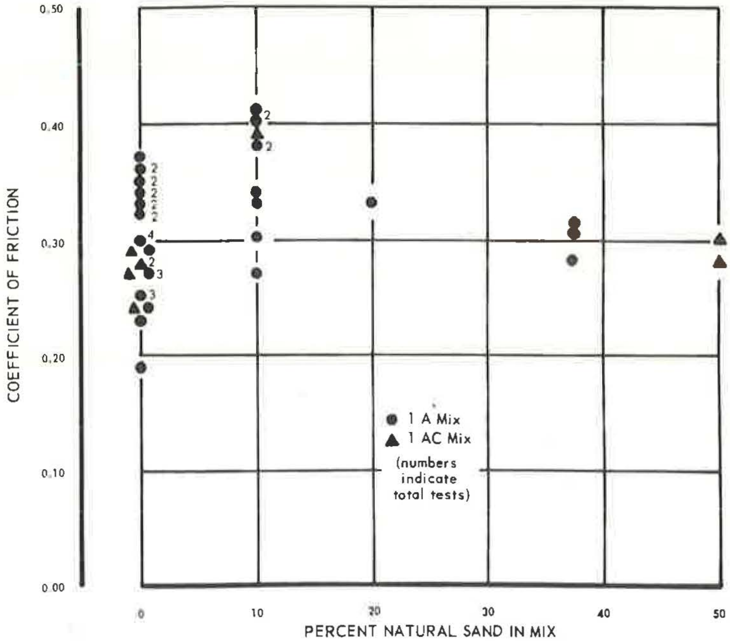

Figure 10. Effect of natural sand in mix (percent sand based on total aggregate weight).

containing limestone coarse aggregate with varying amounts of this sand are shown in Figure 10. Using 50 percent sand did not increase skid resistance, when softer coarse aggregate was present in the mix.

In the second study, pavements containing only hard siliceous sand as aggregate (sand-asphalt mixes) retained excellent skid resistance (Table 3). This Long Island sand has over 90 percent feldspar and silica content (Mohs hardness of 6 and 7, respectively). However, additions of only 5 percent crushed dolomitic limestone (Mohs hardness = 3) of No. 1A size (primarily ⅛ to ¼ in.) to the sand mixes in all cases decreased the coefficients of friction. Skid resistance was even lower when pavement contained 50 percent dolomitic limestone and 50 percent sand.

It is apparent that skid resistance of a bituminous mix is governed by its largest aggregate. In New York State 1A and 1AC mixes, the coarse aggregate (or stone) determines the pavement's skid resistance.

TABLE 3
INFLUENCE OF COARSE AGGREGATE ON SKID RESISTANCE

Dolomite Coarse Aggregate Added to Silica Sand	Cumulative Traffic, millions	Coefficient of Friction	
		Test	Avg.
None	6	0.52	0.49
	30	0.45	
	5	0.41	
5 Percent	6	0.39	0.38
	35	0.34	
	35	0.36	
	50+	0.41	
	18	0.32	
	22	0.29	
50 Percent	41	0.29	0.29
	50+	0.28	
	50+	0.27	

Effect of Coarse Aggregate

Having established that skid resistance is governed by coarse aggregate, pavements were evaluated according to the type of stone (over ⅛ in.) they contained, regardless of their fine aggregate. Most pavements containing limestone or dolomite coarse aggregate had friction coefficients below 0.32 after 5 million vehicle passes, although some roads with similar coarse aggregate types remained skid resistant after more than 10 million vehicle passes. Stone in the latter pavements was found to contain hard, sand-sized particles embedded in the softer dolomite or limestone matrix. Traffic wore away the soft material, exposing the hard impurities and producing a sandpaper

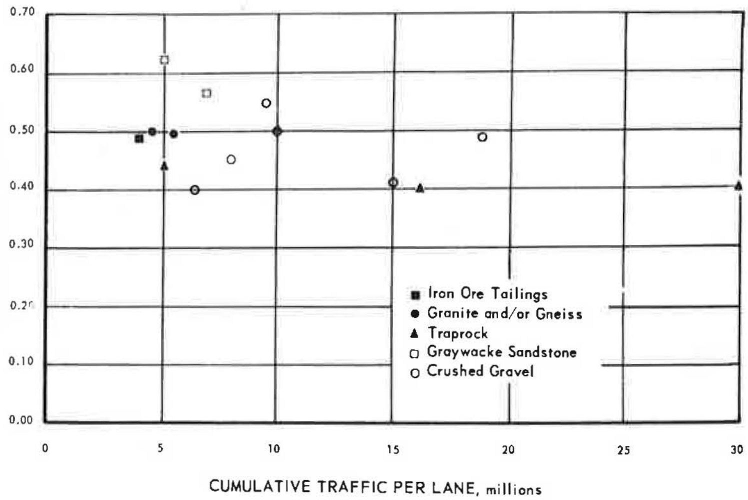

Figure 11. Skid resistance of bituminous-concrete surfaces containing hard and angular coarse aggregates.

texture on the coarse aggregate. This differential wear resulted in pavements with high coefficients.

Pavements containing only hard angular aggregate (Mohs hardness = 6+) remained skid resistant after prolonged exposure to heavy traffic. Friction coefficients of 14 such surfaces (Fig. 11), containing traprock, sandstone, crushed gravel, iron ore tailings, or granite as coarse aggregates, remained at or above 0.40 after cumulative traffic in excess of 4 million vehicles.

Skid resistance was primarily controlled by the ability of larger aggregates to resist traffic's polishing action. To evaluate wearing characteristics of certain New York State aggregates of known skid resistance, Los Angeles, Deval, and Dorry abrasion tests were run[1]. Figure 12 shows coefficient of friction as a function of percent loss in these various abrasion tests. Statistical analysis of the data provides 99 percent confidence that no correlation existed between coefficient of friction and loss in any of the three abrasion tests.

In January 1965, Gray and Renninger (6) reported an acid leaching procedure for evaluating skid resistance properties of carbonate aggregates. Aggregates were allowed to react with dilyte hydrochloric acid until all carbonates had dissolved. The percentage by weight of hard, acid-insoluble, sand-sized particles was then determined. About 10 percent hard impurities in the soft matrix significantly affected skid resistance.

Skid tests were conducted in the summer of 1965 on 31 New York State highways having limestone, dolomite, or traprock aggregate with large cumulative traffic volumes. Aggregate was extracted from samples taken from each surface. Acid leaching was performed on only the large stone (over ¼ in. from 1A mixes and over ⅛ in. from 1AC mixes). The acid was changed as needed until all reaction stopped. The residue was washed, and all material passing the No. 200 sieve was discarded. The clay and silt-sized particles either floated away or passed through the No. 200 screen. Remaining particles were found to be mostly silica, with a small quantity of other hard material which was not classified.

[1]Los Angeles: ASTM C 131 (small-sized coarse aggregate)
ASTM C 535 (large-sized coarse aggregate)
Deval: ASTM D 289 (coarse-graded aggregate)
Dorry: USDA Bulletin No. 347 (March 17, 1916).

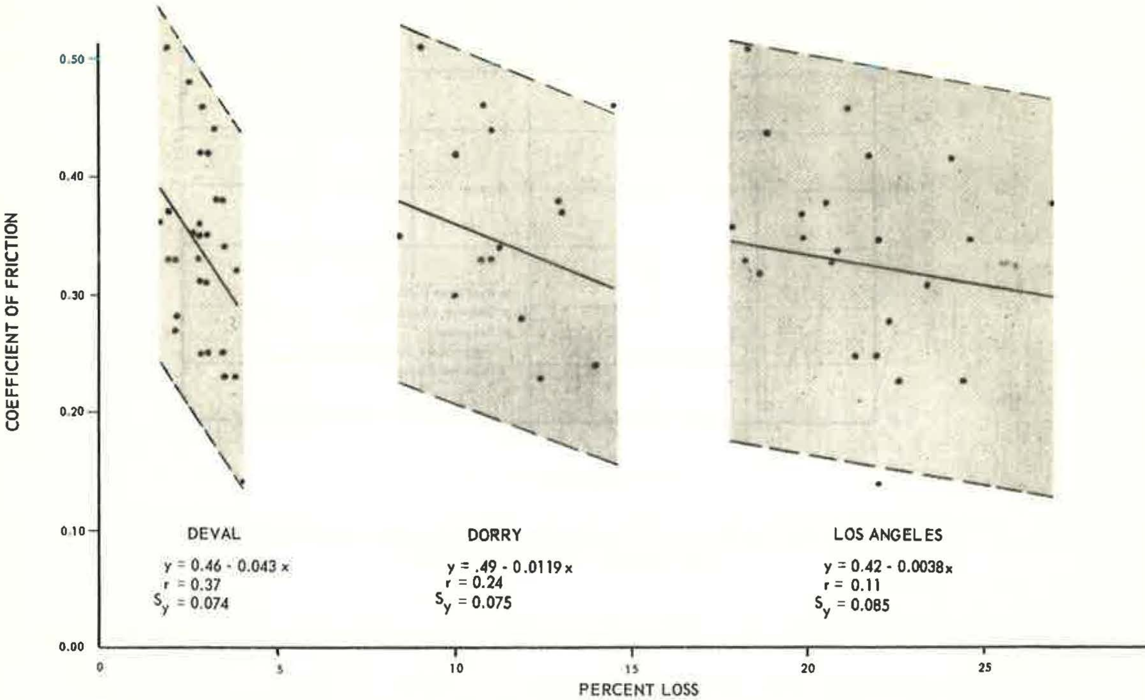

Figure 12. Relationship between coarse aggregate abrasion resistance and pavement friction coefficient.

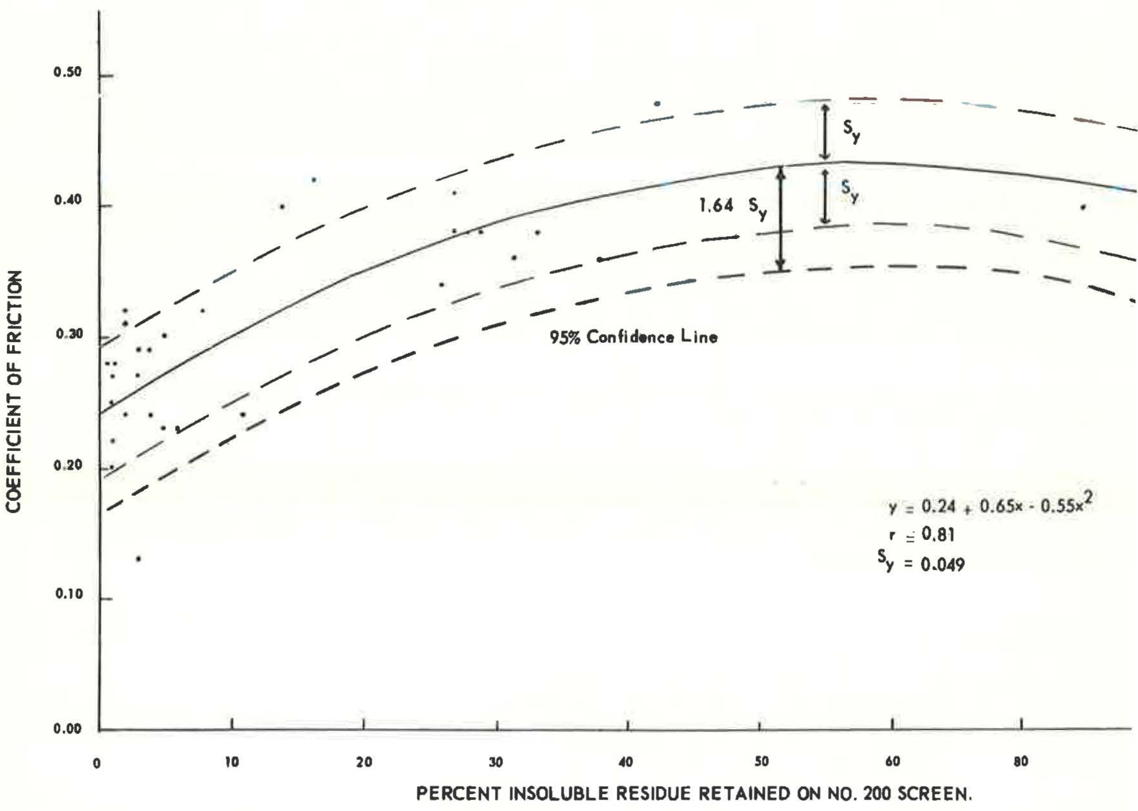

Figure 13. Correlation of pavement skid resistance with acid-insoluble material in coarse aggregate.

The residue was weighed and its percentage by weight computed. The relationship between skid resistance and percent acid-insoluble residue retained on the No. 200 screen is shown in Figure 13.

The coefficient of friction increased with increasing percent insoluble residue in the larger-sized aggregate. For example, reading along the 95 percent confidence line, a pavement with aggregate containing 40 percent insoluble residue should retain a coefficient of friction of 0.33 or more. A good correlation (r = 0.81) was obtained.

Continuation of Field Studies

Experimental surfaces are being placed on State highways to determine the most practical and economical way to construct skid-resistant bituminous-concrete pavements. Surface textures of these experimental pavements range from a fine sheet asphalt to an open-graded asphalt concrete. These surfaces contain varying quantities of hard, acid-insoluble materials and are being placed in areas of high traffic volume to obtain results as quickly as possible.

CONCLUSIONS

1. The New York State skid trailer produces valid skid-resistance measurements, highly reproducible over a wide range of coefficients. This system correlates very well with the stopping distance method.
2. In 2 years of testing, it was demonstrated that new pavement skid resistance declines rapidly at first and then more gradually. After about 5 million equivalent vehicle passes (1 truck = 11 cars), a nearly constant coefficient is reached. Testing is recommended at this level of wear, when long-term skid-resistance properties can be validly measured.
3. Three standard New York State bituminous mix types are very similar in skid resistance characteristics at 40 mph, none showing any superiority to the others.
4. Skid resistance of a bituminous surface is governed by the larger aggregates in the mix, rather than the properties of the fines. Coarse aggregate containing hard impurities resistant to polishing by traffic is superior to that composed entirely of softer material.

ACKNOWLEDGMENTS

This investigation was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads, under the State-Federal Highway Planning and Research program.

The work was performed by personnel of the Bureau of Physical Research, New York State Department of Transportation, Malcolm D. Graham, Director. Many Bureau employees were involved in this study, with the major portion of the field work performed by John L. Whitmore.

New York State Department of Transportation personnel throughout the State were helpful in selecting test sites, supplying plant data, and collecting aggregate and pavement samples.

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Discussion

GLENN G. BALMER, Highway Research Engineer, Bureau of Public Roads — The authors of this paper are to be congratulated for such a realistic investigation and analysis of a timely problem.

It is gratifying to find confirmation of the acid-insoluble residue test as an analysis of the skid-resistant characteristics of aggregates similar to the results obtained in the paper, "Laboratory Studies of the Skid Resistance of Concrete," published in the ASTM Journal of Materials, Vol. 1, No. 3, September 1966.

The residue test is a simple means of determining the skid-resistant quality of the surface aggregates prior to construction of the pavement. This test can be conducted when the friction characteristics of the aggregates are unknown or doubtful, and avoid construction of pavements with inferior friction surfaces and also avoid expensive pavement resurfacing or other treatments to improve skid resistance.

Laboratory Considerations for the Use of Lightweight Aggregates for Hot-Mix Asphalt Pavements

BOB M. GALLAWAY and WILLIAM J. HARPER, Texas Transportation Institute, Texas A & M University

A literature search revealed only limited material published on the use of lightweight aggregate in plant-mixed asphalt pavements. Data from eight different trial sections indicate that such materials produce paving mixtures of acceptable quality.

Class I synthetic aggregates from seven sources were used with two different field sands to produce laboratory designs that were tested by several methods to determine their suitability for hot-mix asphalt pavements. Limited laboratory studies of the following parameters were included for specific mixtures of synthetic aggregate, field sand, and paving grade asphalt cement: (a) laboratory compaction degradation, (b) Hveem stability and cohesion, (c) asphalt demand, (d) water susceptibility, (e) swell characteristics and expansion pressure, and (f) air permeability.

Compaction degradation was measured on one material for a 100 percent synthetic aggregate design by examining the change in the particle size distribution after laboratory compaction at three energy levels and four asphalt contents. An analysis was also made on synthetic aggregate from all sources for designs containing field sand. Asphalt content was varied and change in the surface area of the aggregate in each design was measured.

Hveem stability and cohesiometer measurements were made on designs involving synthetic aggregate from all sources. Asphalt content was varied from 6.0 to 10.0 percent for all designs.

The study included laboratory measurements of asphalt absorption as determined by examining typical mix designs and by complete immersion of the synthetic aggregate in hot asphalt cement. Comparisons of asphalt and water absorption were made. Data on total asphalt demand are included.

Water susceptibility of selected hot-mix designs was determined by the immersion-compression test (ASTM 1074-60). Swell characteristics of typical laboratory designs were measured by the Texas Highway Department method (Test Method Tex-209-F). Expansion pressure was measured after the method of the California Department of Highways.

Air permeability was measured on designs made from aggregates of the different sources. A range of asphalt contents and air voids was included.

•SINCE the introduction of lightweight synthetic aggregate as coverstone for seal coats and surface treatments on Texas highways in 1961, aggregate producers, contractors,

Paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction and presented at the 47th Annual Meeting.

highway personnel, and even the driving public have watched the performance of this material with a critical eye. Service records for the past 5 years are now available and these records show conclusively that synthetic aggregate of the proper quality produces a high-performance coverstone provided proper procedures are observed in the design and construction of such surfaces. Records showing that this material is serving the driving public safely and economically are available on some 2000 miles of primary and secondary Texas highway with traffic volumes from 100 to 8000 vehicles per day.

It seemed reasonable to expect that this same type of material would serve equally well in hot-mix asphalt paving materials; therefore, this exploratory research was undertaken to verify this hypothesis.

The basic physical characteristics of synthetic aggregates that definitely influence the use of these materials in asphaltic concrete are asphalt affinity, abrasion or wear characteristics, and aggregate durability as determined by freezing and thawing or sodium sulfate soundness. Data on these properties are given in papers by Gallaway and Harper (1) and by Ledbetter (2).

The research approach for verification of this hypothesis was a complete factorial design including the necessary basic research, laboratory evaluations, and field service trials. This study, however, covers only a limited segment of the overall research plan and is therefore incomplete and the conclusions are tentative. It is, nevertheless, clearly evident that synthetic aggregate has a definite potential as a major portion of the aggregate system in flexible pavement structures.

All of the synthetic aggregate can be classified as Class I Group A, or Class I Group B, according to the proposed synthetic aggregate classification system (3).

OBJECTIVES AND PLAN OF RESEARCH

This research was conducted to determine the basic physical characteristics of synthetic (lightweight) aggregates and to relate their uses in hot-mix, hot-laid asphalt pavement surfaces. The secondary objective, an outgrowth of a previous study (1), was to determine the physical characteristics of synthetic aggregates affecting their use as aggregate in plant-mixed asphaltic concrete for thin overlays and anti-skid pavements. From this, it was anticipated that guidelines for the design and specification of asphaltic concrete using these aggregates could be produced.

The basic research plan was to examine mixtures containing lightweight aggregate and to consider the following:

1. Laboratory compaction degradation;
2. Hveem stability and cohesion;
3. Asphalt demand by film thickness;
4. Water susceptibility;
5. Swell characteristics and expansion pressure; and
6. Permeability to air.

A limited study of these items was conducted in the laboratory to examine certain design parameters. Thus far, there has been no correlation of these data to the field performance of asphaltic-concrete mixtures made from synthetic aggregate blends.

BACKGROUND

In 1955, Louisiana placed a field test section of asphalt pavement made from lightweight aggregate hot-mix. This experimental section of roadway was 200 ft long and 4 traffic lanes wide. The compacted layer was approximately 2 in. thick (4). The lightweight aggregate was an expanded clay from the same source as one of the materials studied in this investigation.

The Louisiana study incorporated the lightweight aggregate as the material coarser than a No. 40 sieve. The mixture design (Marshall method) included fine river sand for the aggregate passing the No. 40 sieve, and the asphalt content was 12 percent by weight on an 85-100 penetration grade asphalt (4). The road was in good condition at the time of reporting (1959), with a daily traffic volume of 7300 vehicles.

In 1955, the Southern Lightweight Aggregate Corporation also became interested in the potential use of lightweight aggregate for asphaltic-concrete surfaces. As reported by Wycott (5), their study included a design by the Hubbard-Field method and strength testing by the ASTM methods D 1074-60 and D 1075-54. The aggregate used was 100 percent lightweight aggregate, and the grading was the same as that for concrete masonry units (ASTM C 331-53T). Bitumen contents ranged from 9 to 12 percent by weight for the laboratory test. In 1957, a field trial of the optimum laboratory design was made in Richmond, Va. This test section was also 200 ft long and 4 traffic lanes wide. The gradation of the lightweight aggregate in the field trial was changed slightly from the laboratory design and the asphalt content was 11.2 percent by weight. The pavement, which had an average daily traffic of 12,700 vehicles, was in excellent condition two years later (5).

Texas, a leader in the use of lightweight aggregates for seal coats and surface treatments (1), has also placed some experimental pavement surfaces utilizing synthetic aggregates. The State's first experimental section of synthetic aggregate was constructed on SH-6 in Fort Bend County in August 1963. The aggregate blend was approximately 68 percent calcined clay and 32 percent field sand and the asphalt content was 6.2 percent by weight of the mixture. The laboratory compacted specimens made from samples of loose mix secured from the field had an average Hveem stability of 41 percent and 3.4 percent air voids.

Since 1963, several districts of the Texas Highway Department have made laboratory and field trials using synthetic aggregates in hot-mix asphaltic-concrete surfaces and bases, but detailed reports have not been published on these trials. The most recent of these field trials was on I-20 near Mesquite on the inside lane of the Dallas-bound roadway. Average daily traffic in 1965 was approximately 33,000 vehicles. Two different sources of lightweight aggregate were used and both materials were evaluated to a limited extent in this research. Mixture designs were made using the Texas Highway Department modification of the Hveem method; laboratory designs yielded stabilities in the order of 45 to 50 percent and cohesiometer values of 100 to 150 gr per inch of width. The air voids of the laboratory specimens were approximately 2 to 5 percent. The field test included two designs for each aggregate; however, both designs used 50 percent by weight lightweight aggregate and 50 percent sand. The basic difference in each design was the type of sand used. The asphalt content in all four sections was 7.0 percent by weight of mixture. These pavements have been in service about 17 months and are performing satisfactorily. The Texas Highway Department measured the skid properties or coefficient of friction of these surfaces after about 15 months of service. The coefficient of friction of the lightweight sections averaged about 0.48 at 50 mph, while that of the adjacent lane, placed at the same time but utilizing normal aggregates, was 0.31.

MATERIALS

Lightweight Synthetic Aggregate

The aggregates were secured from the six producers of lightweight aggregate in Texas and from one producer in Louisiana. These included both expanded clay and expanded shale products and fell into Class I of the proposed THD classification system for synthetic aggregates. The materials from all of the potentially available suppliers were used because each supplier uses different raw material and different methods of burning and crushing, or both. Hence, these aggregates represent the entire range of such materials currently produced in Texas.

The major interest of each of these producers is the production of aggregate suitable for use in the concrete block industry; therefore, the materials supplied did not conform to the grading requirements of Texas Highway Department specifications (1962) for asphaltic concrete. However, production procedures should be adaptable to grading requirements.

Aggregates used were the same as those in the preceding phase of the study (1). In general, they were Type F, Grade 3 or 4, conforming to Texas Highway Department

TABLE 1

PHYSICAL PROPERTIES OF LIGHTWEIGHT AGGREGATES

Aggregate	Raw Material	Vacuum Sat. Density (% in.-No. 4, g/cc)	Dry Loose Unit Wt.[a] (pcf)	Los Angeles Abrasion (C Grading), % Loss	
				ASTM	THD Item 1269
A	Shale	1.84	45.7	23.8	17.8
B	Shale	1.42	40.6	25.0	15.3
C	Clay	1.35	41.3	24.4	13.9
D	Shale	1.68	48.9	22.0	14.8
E	Clay	1.62	38.6	34.9	40.7
F	Clay	2.01	43.7	28.6	20.0
G	Shale	1.77	45.5[b]	25.4	21.2

[a]THD Item 1269, Grade 4.
[b]THD Item 1296, Grade 3.

Special Specification, Item 1269, Aggregates for Surface Treatments (Lightweight). Typical physical properties are given in Table 1. Since these aggregates are generally produced for concrete block and sealcoat work, it was necessary to screen and grade them to meet the mixture design requirements.

The lightweight aggregate was used as the coarse fraction (plus No. 10 sieve) of the asphaltic-concrete surface course to provide better skid-resistant properties in the pavement. According to Kenneth Hawkins, field measurements show that lightweight aggregates used in this manner do not polish or become slick—as they wear a textured surface will remain. Also, the low unit weight property of the material was used to maximum advantage, thus effecting greater economy in the design.

Field Sands

Since the lightweight material is the coarse fraction, the fine fraction should consist of some locally available filler material. This would normally consist of field sand, crusher screenings, shell, or possible lightweight fines; however, the use of a lightweight fine fraction would increase the asphalt demand, arising from the increased volume for a given unit of weight for the lightweight fines. In addition, the lightweight fine aggregate is more absorptive than most stone screenings.

Field sand was chosen for this study because of its economy and wide availability. It is normally expected to provide the particle sizes smaller than the No. 8 sieve. In some instances, as was the case in this study, a blend of a coarse and fine sand may be necessary to obtain an improved particle-size distribution. The sieve analysis data for the field sands are given in Table 2. These sands, typical of many sands found in Texas, are designated FS 1 and FS 2.

Asphalt

The asphalt was an 85-100 penetration grade with an intermediate susceptibility to hardening. This asphalt would be classified as to viscosity as between an AC-10 and an AC-20, according to present Texas specifications (Table 3).

This asphalt was used throughout the study so that the type and grade of binder would be constant and it was chosen as representative of the asphalts commonly used for surface courses of asphaltic concrete in Texas.

TABLE 2

GRADATION OF FIELD SANDS

U. S. Sieve No.	Percent Passing	
	Field Sand No. 1	Field Sand No. 2
16	100.0	100.0
30	99.4	100.0
50	67.8	98.7
100	17.4	75.9
200	8.4	28.5

TABLE 3

ASPHALT-CEMENT CHARACTERISTICS

Viscosity	
At 77 F and Sr + 5 × 10^{-2} sec^{-1}, poise	1,020,000
At 140 F, poise	1,760
At 275 F, poise	2.75+
Penetration, 100 g, 5 sec, 77 F, points	90
Specific gravity, 77 F/77 F	1.014
Ductility, 5 cm/min, 77 F, cm	150+

DESIGN DATA

Gradation

The Texas Highway Department uses a modification of the Hveem procedure for its design and control work. Modifications are essentially in the area of predicting an optimum asphalt content and in molding of the test specimens. The procedure involves the use of a gyratory shear-type molding press for forming both the laboratory and quality control specimens.

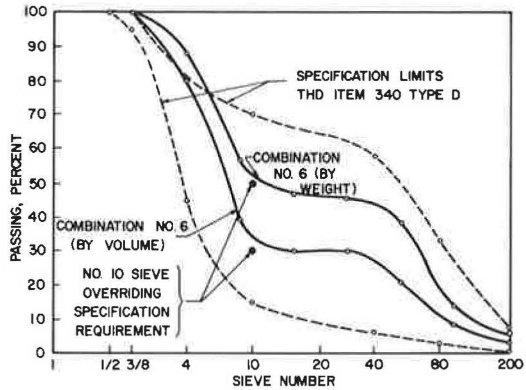

Figure 1. Aggregate blends by weight and volume methods.

One of the primary considerations in the design of an asphaltic-concrete mixture is the gradation requirements. The aggregate blend may vary from a dense combination of materials to a gap or skip gradation. Texas specifications for aspahltic concrete lend themselves to the latter type. This also proves to be advantageous in the design of mixtures utilizing lightweight aggregate because the lightweight material is generally used as the coarse fraction (plus No. 10 sieve) and is shipped to the job site, whereas the fine fraction may be a locally available field sand which would introduce a gap in the gradation. The use of gap graded blends containing lightweight aggregate is generally satisfactory since their stability will nearly always meet specified requirements and will probably be workable in the field.

Unit weight is another major factor in blending lightweight aggregates. Normally, lightweight aggregate will have a dry loose unit weight in the range of 35 to 55 pcf, whereas the sand or normal weight aggregate will have a dry loose unit weight of 90 to 100 pcf. This difference can result in serious difficulty if it is not considered when making the aggregate combination (Fig. 1). It is necessary to combine the materials on a volume basis and convert the combination to weight measurements for field-batching purposes. Weight measurements are more accurate and are easily controlled in both the laboratory and the field.

A number of aggregate blends were considered before a selection was finally made (Fig. 2). Combination No. 2 is a dense-graded blend containing approximately 70 percent lightweight aggregate by volume. It was seriously considered but was not used in the study because it was believed that the asphalt demand would be excessive, since approximately 20 percent by volume of the lightweight fraction was between the No. 10 and the No. 30 sieve. Also a grading of this type might not be as economical as blends containing local materials. Combinations No. 4 and No. 6 were then considered as being the logical choices economically. Combination No. 4 containing 50 percent lightweight coarse aggregate (by volume) would be the most adaptable for field uses because of improved workability. However, Combination No. 6 containing 70 percent (by volume) lightweight aggregate was chosen because it represented the maximum probable amount of lightweight material that could be incorporated in a bituminous mixture. It

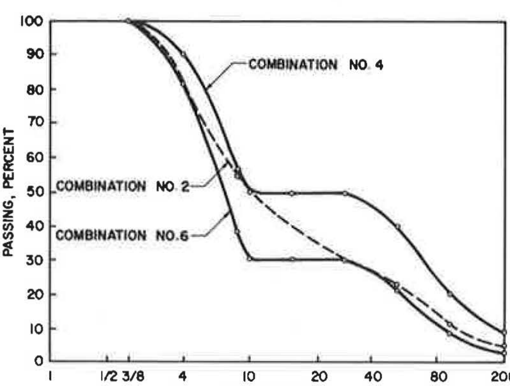

Figure 2. Gradation chart for various aggregate blends (volume combination).

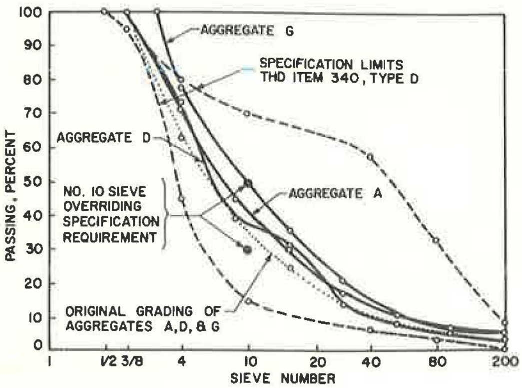

Figure 3. Original and recovered aggregate from a degradation study of 100 percent LWA mixtures.

was considered that this maximum blend would produce the most unfavorable conditions if the synthetic aggregate were not suitable for asphaltic concrete.

Laboratory Compaction Degradation

One of the more important problems of the study was in laboratory compaction-degradation; hence, such a study was undertaken. A dense-graded combination containing 100 percent lightweight aggregate was selected, for two reasons: (a) an all-lightweight design would be most susceptible to particle breakdown, and (b) any added fine material such as field sand would cloud any analyses made with sieves. To determine the degrading effect by some other method would be more expensive. The original grading curve for this combination is shown in Figure 3.

The mixtures were prepared at three asphalt contents with the estimated optimum value being that determined by the California centrifuge kerosene equivalent method (6). Design details are given in Table 4. These mixtures were prepared by the Texas gyratory shear method in accordance with standard procedure (7). Various laboratory tests outlined in the plan of research were performed on the test specimens and the asphalt was extracted from the aggregate by reflux extraction (AASHO T184 60). A sieve anaylsis was made on the recovered aggregate to determine the change in particle size distribution. The surface area (6) was also computed and these data are tabulated in the Appendix. Typical data are shown in Figure 3.

TABLE 4

TYPICAL MIXTURE DESIGN DATA FOR LABORATORY COMPACTION DEGRADATION

(100 Percent Lightweight Aggregate A)

Asphalt Content	Density g/cc	Voids %	Stability %	Cohesion g/in. width
8.0	1.50	10.7	46	61
9.0	1.51	9.3	47	94
10.0	1.54	6.3	48	132

There was no pattern of behavior or relation between the effects of asphalt content and the change in surface area. Data for these aggregates are given in Table 5. The differences in the original and final surface areas do not reflect which original particles received the most damage during compaction. For example, aggregate G has a smaller change in surface area, but the particles between the $^3/_8$ and $^1/_4$-in. sieves have disappeared (Fig. 3). The possible relationship between the Los Angeles abrasion test and the change in surface area was examined and no positive correlation was found to exist.

TABLE 5

EFFECT OF ASPHALT CONTENT ON CHANGE IN SURFACE AREA

Aggregate Source	Asphalt Content % by Weight	Change in Surface Area, % of Original
A	8.0	33.1
	9.0	34.6
	10.0	42.8
D	5.0	24.4
	6.0	4.5
	7.0	4.0
G	8.0	15.2
	9.0	28.9
	10.0	25.1

In addition, sieve analyses were also made on the recovered aggregate from hot-mix designs containing lightweight aggregate and field sand. The existence of errors due to differences in the unit weight of particles on a given sieve was recognized; however, it was felt that these data might still have some value. Since the aggregate blend used in this study was a volume combination, it was converted to a weight basis for a better comparison with the data on the recovered aggregate. Typical results are shown in Figure 4, which includes the original gradation of Combination No. 6 computed on both volume and weight basis and the gradation of the aggregates recovered from hot-mix designs made from field sand and aggregates D and E. Aggregate E had the most laboratory degradation, whereas the best aggregate, D, showed no appreciable breakdown. Figure 5 shows aggregate B broken down into the percent retained between individual sieves. The coarse aggregate was apparently breaking into smaller pieces with only minor changes taking place in the finer material.

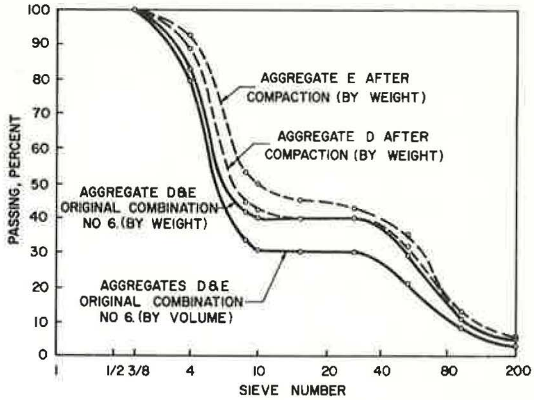

Figure 4. Original and recovered aggregate from a degradation study of LWA and field sand mixture.

The coarse material retained on the No. 4 sieve was reduced approximately 6 percent by weight and the total weight on the Nos. 8, 16, and 30 sieve sizes increased by about the same amount, indicating that the lightweight was degrading. Further examination of the material retained on the No. 50 sieve indicated the possibility of degradation in field sand.

Since there may have been differences in degradation characteristics due to different compactive efforts, a series of laboratory compaction tests was conducted using the Texas Highway Department manual molding press and the motorized press at two energy levels. Results in the form of a sieve analysis are given in Table 6.

These limited data indicate that there was no significant difference in degradation due to compaction in the manual press and the motorized press, or between the various energy levels of the different presses for aggregate A. It cannot, however, be assumed that this resistance to degradation would prevail for other synthetic aggregates produced in Texas.

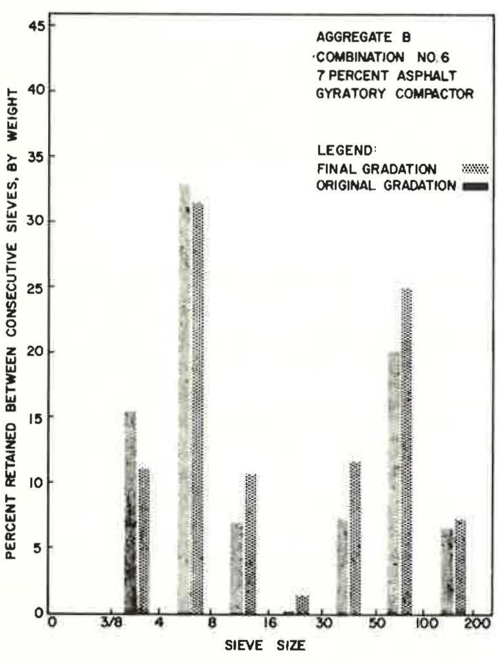

Figure 5. Aggregate degradation for individual sieves.

Strength Measurements

Texas uses a modification of the California design procedure for establishing

TABLE 6

EFFECT OF COMPACTIVE EFFORT ON COMPACTION DEGRADATION

(Aggregate A, Combination No. 6, 6.5 Percent Asphalt)

U. S. Sieve Size	Aggregate Gradation Percent Passing (by Weight)			
	Before Compaction	After Compaction		
		THD Manual Press	THD Motorized Press	
		100-psi End Point	50-psi End Point	150-psi End Point
⅜ In.	100.0	100.0	100.0	100.0
No. 4	83.6	83.7	83.3	82.8
No. 8	43.9	50.4	50.2	48.9
No. 16	36.3	37.8	37.3	37.4
No. 30	36.1	36.7	36.0	36.1
No. 50	26.5	30.1	30.1	29.4
No. 100	9.9	10.6	10.5	10.5
No. 200	4.2	4.2	4.6	4.2

TABLE 7

TYPICAL STRENGTH AND DENSITY MEASUREMENTS

Aggregate Source	Asphalt Content (% by wt.)	Laboratory Density (g/cc)	Specimen Voids[a] (%)	Stability (%)	Cohesiometer Value (g/in. width)
B	6	1.372	8.3	44	104
	7	1.376	6.3	42	88
	8	1.383	6.1	42	76
	9	1.405	4.4	43	106
D	6	1.681	6.1	42	67
	7	1.696	5.6	40	86
	8	1.717	5.2	40	87
	9	1.741	1.7	37	233

[a] Based on Rice's method for maximum specific gravity (9).

compliance to hot-mix specifications. Current Texas specifications (12) generally require certain density and stability values and, in some cases, cohesiometer values. The aggregates are also required to meet certain grading limits; these requirements for Texas Highway Department Item 340, Type D, are shown in Figure 1.

Stability

The Texas Highway Department modified Hveem stability requirement for most surface course designs is a minimum of 30 percent, and surfaces designed in the normal manner using lightweight aggregate as the coarse fraction will easily meet this requirement. In fact, stabilities of 40 to 50 percent are common. The stability of such asphaltic surface mixtures is generally not very susceptible to change in asphalt content, for example, in the range of 1 or 2 percentage points. This is particularly advantageous because larger amounts of asphalt cement may be incorporated into the mixture for greater durability. Job control is not critical for mixtures containing lightweight aggregate because small variations in asphalt content will not produce unstable mixes, whereas a variation of 0.2 percent asphalt in a slick pea gravel-sand mixture may lead to drastic changes in stability. Some typical data for stability are given in Table 7. In general, stability will increase with increasing asphalt to an optimum amount and then decrease. This is normal behavior for non-lightweight mixtures. However, for the variations in asphalt content indicated above, the stability is nearly constant, i.e., within the repeatability of the test. The data for all mixtures are given in the Appendix.

Cohesion

Cohesion of mixtures containing lightweight as the coarse fraction generally increases with increasing asphalt content; however, the cohesion is highly influenced by the type, grade, and amount of asphalt cement used. The Texas Highway Department currently requires a cohesiometer value of 100 gr per inch of width when specification Item 346 is used, but this item is not in general use. Typical cohesiometer values are given in Table 7 and the Appendix.

Density

The specimen density and air voids as determined by the Rice method (9) were generally within the ranges specified by the Texas Highway Department. Specimen density increases with increasing asphalt content to the point of flushing or zero voids. In this sense, lightweight aggregate mixtures behave as ordinary "dense rock" mixtures. Air voids in the lightweight aggregate mixtures computed in the manner described in the Texas Highway Construction Bulletin C-14 (10) may exceed the allowable specified values. This problem will be studied by the Texas Transportation Institute in a proposed new program, and it may be that new design criteria are in order. Specimen density and air voids are the most repeatable characteristics thus far encountered in the design of mixtures containing lightweight aggregate as the coarse fraction.

Relative density and air voids computations were based upon the specific gravity of the loose mixture after Rice (9) instead of on the formula considerations of Texas Highway Department Bulletin C-14 (Table 7). This was done because the vacuum-saturation procedure takes into account the absorption characteristics of the aggregates, whereas the formula method does not. Hence, because of the absorptive nature of the lightweight aggregate, it was thought that the vacuum-saturation method of determining the maximum specific gravity of the loose mixture would give superior results. The relative density and air voids computed in this manner will produce values lower than those by methods currently specified, but the relative density will never exceed 100 percent. The methods currently used do not account for asphalt absorption by the aggregate and may lead to unrealistic values of 103 to 104 percent relative density (11). Differences for highly absorptive materials such as synthetic aggregate may be expected to be even greater.

TABLE 8

ASPHALT ABSORPTION FOR LIGHTWEIGHT AGGREGATE (Immersion Method)[a]

Aggregate Source	Absorption, % by Weight of Dry Aggregate	
	5/8-3/8 In.	3/8 In.-No. 10
A	5.4	5.8
B	7.7	5.1
C	7.4	7.5
D	2.0	3.6
E	9.5	7.6
F	13.4	15.4
G	10.1	12.6

[a]Modified from method reported by Goshorn and Williams (13).

Asphalt Demand

The asphalt demand for lightweight aggregate hot-mixed asphalt paving mixtures may be predicted by film thickness and surface area methods together with a knowledge of the aggregate absorption requirements.

Asphalt-Absorption

The effective asphalt film thickness for hot-mixed asphalt pavements in Texas is in the range of 5 to 11 μ (11). The asphalt cement required to coat the aggregate to a given film thickness may be computed by a method outlined by Harper, Jimenez, and Gallaway (12) and based on the surface area concepts of Hveem and the California Highway Department (6). When so computed, assuming effective film thickness of 8 μ, aggregate A, for example, requires approximately 5.6 percent (by weight of aggregate) asphalt cement. It logically follows that greater film thicknesses for a given aggregate gradation will require more asphalt. For instance, if the film thickness is increased to 10 μ, about 6.9 percent asphalt cement is required. These asphalt contents are influenced only by the gradation and density of the aggregates involved and not by the "nature" of the stone. The total asphalt content must take into consideration the absorptive characteristics and surface texture of the aggregates as well as the film thickness requirements.

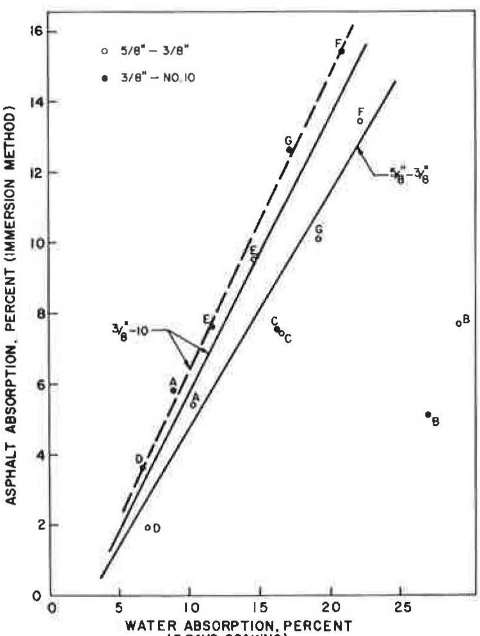

Figure 6. Correlation of asphalt and water absorption.

Since lightweight aggregates have a very porous structure and water absorption values may range up to 30 percent for an aggregate such as B[2], asphalt absorption may be a major factor in mix design considerations; hence, a laboratory study was performed to determine the asphalt absorption characteristics. Two methods for determining the asphalt absorption were used. One was to immerse the aggregate in hot asphalt (13) and determine the absorption when an unlimited supply of hot asphalt was available. The other method was to determine the asphalt absorption from a regular mixture of asphalt and aggregate. The latter approach will limit the asphalt available for absorption and thus decrease the total absorption. These laboratory studies have shown that very good mixtures can be made with lightweight aggregates used as the coarse fraction in spite of the relatively high absorption that takes place.

The method involving the total immersion of aggregate particles into hot liquid asphalt cement is a modification of that reported by Goshorn and Williams (13). To carry out these tests, the coarse fraction was divided into two sizes in keeping with the earlier work in lightweight aggregate seal coats (1) and these fractions were tested by procedures outlined by Goshorn and Williams (Table 8).

In the computation it was necessary to compute the bulk specific gravity of the stone in both the dry and saturated surface dry condition; hence, the water absorption (three days' soaking) was also determined. When the water absorption data were plotted (Fig. 6), it was found that a definite correlation (coefficient of determination, $r^2 = 0.912$) existed between the two parameters for the ⅝ to ⅜-in. size aggregate. A good correlation, $r^2 = 0.86$, was found for the smaller stone; however, there is one outlying data point. If this point were not considered, the line fit would be excellent, as is indicated by the dashed line on the graph. Data for aggregate B were excluded from the regression analysis because production methods subsequently were changed to reduce water absorption.

Probably the most realistic method for obtaining the asphalt absorption was that outlined by Rice (9). This method is preferred since the absorption is calculated from data on actual mixtures. Data in Table 9 are based on the assumption that absorption in the sand is negligible with primary absorption by the lightweight aggregate. Table 9 also includes data from mixtures cured by different methods to determine the effects of time and temperature upon absorption.

The mixtures were made at 9.0 percent asphalt (by weight of mixture) which allows a reasonable amount of asphalt cement available for absorption. The curing times were chosen to represent the maximum time and temperature conditions (curing No. 1) of field mixtures and those more representative of a newly constructed pavement (curing No. 2). The data indicate that regardless of curing conditions, asphalt absorption is almost constant, i.e., approximately 2.0 to 3.0 percent by weight of aggregate.

Total Asphalt Demand

The total asphalt required in a hot-mixed asphaltic-concrete mixture is the sum of the components. For example, it was shown that the amount of asphalt cement needed to satisfy a film thickness requirement of 8 μ was 5.6 percent for aggregate A, and the absorption was 2.4 percent (Table 9). Hence, the total asphalt cement required to make a hot-mixed asphaltic concrete mixture using aggregate A to meet the grading requirements was 8.0 percent by weight of aggregate. This volume may be a little low to meet other specification requirements, but it is a good starting point. The computed asphalt demand was on a weight basis, which is more convenient for batching operations. However, one must also consider the volume of the mixture, and possibly consideration should be made for increasing asphalt content on a volume basis. Research in this area is continuing with the objective of producing design criteria and construction guides for utilizing this material in successful hot-mixed asphalt pavements.

TABLE 9

ASPHALT ABSORPTION FOR LIGHTWEIGHT AGGREGATE[a]

Aggregate Source	Absorption, % by Weight of Lightweight Aggregate in Mix ⅜ In.-No. 10	
	Curing No. 1[b]	Curing No. 2[c]
A	3.1	2.4
B	0.8	2.2
C	2.6	2.2
D	0.4	0.1
E	2.8	2.6
F	2.8	2.0
G	2.8	3.2

[a]Using method reported by Rice (9).
[b]Curing No. 1: 3 hr at 250 F.
[c]Curing No. 2: 1 hr at 250 F and 20 hr at 140 F.

Water Susceptibility

Hot-mixed asphaltic-concrete mixtures made with lightweight aggregate may be susceptible to water, since water absorption of these aggregates is quite high. Without the proper asphalt coating a loss in strength may occur. A study was made of the water susceptibility for the 70 percent lightweight aggregate and 30 percent field sand combination. Mixtures were made at two asphalt contents: (a) a high asphalt content of about 9 or 10 percent by weight of mixture, and (b) a low asphalt content of about 6 or 7 percent. These values were chosen to include the complete range of practical field mixtures. The samples were prepared and tested in accordance with procedures of the ASTM D 1074-60 and D 1075-54. The minimum recommended index of retained strength is 70 percent.

Results of these tests are shown in Figure 7. Aggregates A and D had lower asphalt and water absorption values and a higher index than the center group. Aggregates C, E, F, and G had intermediate water absorption and higher asphalt absorption and tended to fall into one grouping. Aggregate B had a very high water absorption and the slope of the curve was significantly different from the other five groupings. Hence, the asphalt and water absorptions appeared to influence the strength indexes. A direct comparison of the strength index and water absorption was made, but no correlation was found to exist. Based on the strength index criteria, the low absorption aggregates will produce the necessary retained strength at reasonable asphalt contents to make good mixes (Fig. 7). An asphalt content of approximately 9 percent is required for aggregates C and G; however, the entire problem may not lie with the lightweight aggregate. It was observed in the vacuum saturation procedure for specific gravity (9) evaluation of the loose asphalt aggregate mixture that the field sands had a tendency to strip, which might have contributed to the low values of retained strength. Additional research must be carried out with fine aggregate that is not water susceptible, since none of the tests thus far indicate such a weakness in the synthetic material.

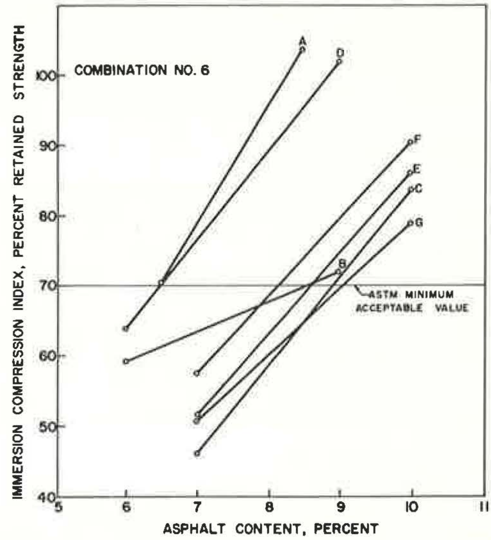

Figure 7. Immersion compression index vs asphalt content.

Expansion Pressure

Another phenomenon related to the introduction of water into the pavement and lightweight aggregate, or both, is that of swell and expansion of the compacted hot-mixed asphaltic concrete. Consideration was first given to the expansion pressure of the molded mixtures, then the swell characteristics were studied to determine if there were any detrimental effects from water.

A test used by the California Highway Department (6) to measure the expansion pressure was used. This is primarily a test for soil samples, but it was considered a reasonable method for determining the swell or expansion of compacted bituminous mixtures. A restrained specimen was soaked in the test device for a period of 24 hr and the upward force or expansion pressure was determined. The bituminous mixtures made from Combination No. 6

TABLE 10

AIR PERMEABILITY OF LIGHTWEIGHT AGGREGATE MIXTURES

Aggregate Source	Asphalt Content (%)	Air Voids (%)	Air Permeability[a] (ml/min)
B	6	8.3	398
	7	6.3	291
	8	6.9	233
	9	4.4	122
D	6	6.1	236
	7	5.6	259
	8	5.2	227
	9	1.7	169

[a]4-in. diameter area; pressure differential 0.25 in. water.

(70 percent lightweight aggregate) and 8 percent asphalt cement yielded no measurable expansion pressure. As a further check on the expansion, aggregate B (highest water absorption) was tested for 120 hr. There was no expansion for the first 72 hr, and the maximum expansion pressure at the conclusion of the test was 1.3 psi.

Swell Characteristics

The swell test for bituminous mixtures THD Test Method Tex-209-F (7) was then used to ascertain if any of the lightweight aggregate mixtures possessed undesirable swell characteristics. The maximum swell permitted by the Texas Highway Department specifications as determined by the change in height of a confined specimen is 0.03 in. Asphaltic concrete with this value or less is considered to have a quality that will resist softening or disintegration when subjected to water. The maximum swell of any of the lightweight aggregate hot-mixed asphalt paving mixtures was 0.004 in. Results of these tests indicate that hot-mix asphaltic concrete made with these synthetic aggregates has exceptionally low swell characteristics.

Permeability

The air permeability of the lightweight aggregate mixtures (Combination No. 6) was studied with the hope that such data could be related to the specimen density (Table 10). In general, air permeability increases with increasing air voids, but the coefficient of determination of such a relationship is 0.43, indicating that no definite correlation exists. In other words, as asphalt content increases, air permeability will generally decrease. The equation used to obtain the values in Table 10 and the Appendix is as follows:

$$K = \frac{u\,Q\,L}{A(P_1 - P_2)}$$

where u = viscosity of the air, Q = rate of flow, L = height of sample, A = area, and P_1-P_2 = pressure difference. The air permeability of these mixtures is very erratic Both the reproducibility and repeatability are not very good.

The air permeability apparatus used in this study is manufactured by Soiltest, Inc., under license from the California Research Corporation. The use of this equipment was first described by Ellis and Schmidt (14) and later by Hein and Schmidt (15). The particular testing procedure used in this study is that supplied by the manufacturer of the apparatus for testing 4-in. diameter laboratory test specimens.

SUMMARY AND CONCLUSIONS

The following summary of results and conclusions are tentative since they are based on limited laboratory data and field trials:

1. Research findings reveal that economical hot-mix designs can be produced by blending synthetic coarse aggregate (½ in. to No. 10) with locally available fine aggregates such as crusher fines and field sand, or both. Where field sand alone is used as the fine material, the coarser gradings produce more economical mixes.

Designs meeting the specification requirements of the Texas Highway Department's Item 340, Hot-Mix Asphaltic Concrete Pavement, Class A Type D, were easily obtained with the materials under study. Proof of service performance for three producers' products has been obtained.

2. Laboratory compaction degradation was found to be a minor problem even for designs containing 100 percent lightweight aggregate. The Texas gyratory shear compactor was used in the study; it is not known what results would be obtained with the Marshall impact hammer or the California kneading compactor. A high Hveem stability is a common characteristic of designs containing aggregate with a rough surface texture and it is probably for this reason that the hot-mix designs produced stabilities in the range of 40-50. Large changes in asphalt content had little effect on measured stabilities. This characteristic has service advantages and economic potential.

3. Asphalt absorption of the synthetic aggregate was essentially constant at 2 to 3 percent for the various producers' products when the available asphalt was limited; however, when an unlimited supply of hot asphalt cement was made available to the different materials, considerable difference was noted in the absorption capacity. Depending on particle size distribution and source of material, the absorption varied from 2.0 to 15.4 percent by weight. Under plant and field construction conditions, asphalt absorption of the synthetic aggregate fraction would normally be in the range of 2 to 3 percent by weight. Microscopic examinations indicate this absorption to be nonselective. Design asphalt contents of 7 to 10 percent by weight of mix are common. Corrected to a volume or film thickness basis, these compare favorably with THD Class A Type D hot-mix dense aggregate designs in use today.

4. Hot-mix designs examined for water susceptibility included field sands; the method used to make the evaluations is not absolute. However, at reasonable asphalt contents most of the designs were acceptable from the standpoint of water susceptibility.

5. The lightweight aggregates exhibited negligible expansion pressure and the swell as measured by Test Method Tex-209-F was in the range of 0.004 in. or less, compared to an allowable of 0.03 in. Therefore, the qualities measured by these tests were quite high.

6. Air permeability measurements were made on a single design using aggregates from all seven sources. As has been found in the past, a general decrease in air permeability is associated with an increase in asphalt content; however, a coefficient of determination of 0.43 was obtained when air permeability was related to air voids in the compacted laboratory specimens.

7. Since most lightweight aggregate particles are highly textured, workability of plant mixed designs containing these aggregates should receive special attention.

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Appendix

DATA SUMMARY
LIGHTWEIGHT AGGREGATE (LWA) MIXTURES

Source of Material	Aggregate Combination	Asphalt Content (%)	Specimen Density (g/cc)	Maximum Density[a] (g/cc)	Air Voids[a] (%)	Theoretical Density (g/cc)	Theoretical Air Voids (%)	Air Permeability (ml/min) of Water[b]	Stability (%)	Cohesion (g/in. width)	Strength Index ASTM 1075-54 (%)	Original Surface Area (sq ft/lb) Volume Basis	Original Surface Area (sq ft/lb) Weight Basis	Final Surface Area (sq ft/lb) Weight Basis	Change in Surface Area (% of original)
A	100% LWA	8.0	1.499	1.680	10.7	1.810	17.1	1659	46.5	61	—	19.7	19.7	26.2	33.1
		9.0	1.512	1.667	9.3	1.790	15.5	1339	47.0	94	—	19.7	19.7	26.5	34.6
		10.0	1.543	1.645	6.3	1.780	13.0	334	48.0	132	—	19.7	19.7	28.1	42.8
	No. 6	4.5	1.568	1.849	13.2	2.051	23.5	—	40.6	46	—	26.0	33.6	—	—
		5.5	1.579	1.803	11.4	2.029	22.2	—	40.2	51	—	26.0	33.6	—	—
		6.5	1.642	1.789	8.2	2.008	19.4	—	41.0	77	70.5	26.0	33.6	35.5	5.6
		7.5	1.637	1.773	7.7	1.987	17.6	—	41.3	82	—	26.0	33.6	—	—
		8.5	1.653	1.765	6.3	1.966	15.9	—	40.0	101	103.5	26.0	33.6	—	—
	No. 4	6.5	1.788	1.977	9.6	2.250	20.6	—	36.0	110	—	53.5	53.5	—	—
	No. 2	7.5	1.702	1.830	6.9	—	—	—	42.0	99	—	34.3	34.3	—	—
B	100% LWA	9.0	1.028	1.248	17.6	1.437	28.5	4041	49.9	67	—	19.7	19.7	22.3	13.1
		10.0	1.080	1.227	12.0	1.431	24.5	2475	49.5	73	—	19.7	19.7	25.9	31.8
		11.0	1.087	1.223	11.1	1.425	23.7	1368	48.0	133	—	19.7	19.7	28.4	44.3
	No. 6	6.0	1.372	1.496	8.3	1.693	19.0	398	43.8	104	59.3	28.0	40.7	42.0	3.1
		7.0	1.376	1.469	6.3	1.682	18.2	291	42.0	88	—	28.0	40.7	43.8	7.6
		8.0	1.383	1.486	6.9	1.669	18.2	233	42.0	76	—	28.0	40.7	42.5	4.5
		9.0	1.405	1.470	4.4	1.658	15.3	122	43.2	106	72.1	28.0	40.7	42.7	4.9
C	No. 6	6.0	1.316	1.462	10.0	1.659	20.7	261	47.0	65	—	27.9	41.4	44.1	6.5
		7.0	1.342	1.437	6.6	1.648	18.6	165	45.0	76	46.3	27.9	41.4	46.7	12.8
		8.0	1.348	1.418	4.9	1.628	17.2	95	44.0	67	—	27.9	41.4	49.4	19.3
		9.0	1.364	1.413	3.5	1.626	16.1	152	46.0	81	—	27.9	41.4	45.3	9.3
		10.0	1.362	1.411	3.5	1.615	15.7	227	45.0	104	83.6	27.9	41.4	41.8	1.0
D	100% LWA	5.0	1.374	1.688	18.5	1.715	19.8	2250	44.3	33	—	19.7	19.7	24.5	24.4
		6.0	1.408	1.674	15.9	1.702	17.4	3002	44.0	50	—	19.7	19.7	20.6	4.5
		7.0	1.411	1.672	15.6	1.690	16.5	3240	46.5	64	—	19.7	19.7	20.5	4.0
	No. 6	6.0	1.681	1.791	6.1	1.872	10.2	236	41.5	67	64.1	27.7	36.9	37.1	0.4
		7.0	1.696	1.824	5.6	1.855	8.6	259	40.0	86	—	27.7	36.9	37.8	2.4
		8.0	1.717	1.811	5.2	1.839	6.6	227	40.0	87	—	27.7	36.9	37.1	0.4
		9.0	1.741	1.771	1.7	1.822	4.4	169	36.5	233	101.8	27.7	36.9	38.0	3.1
E	100% LWA	8.0	1.347	1.572	14.3	1.599	15.8	2423	45.3	80	—	19.7	19.7	31.3	58.8
		9.0	1.337	1.561	14.3	1.589	15.9	2888	44.8	99	—	19.7	19.7	28.8	46.4
		10.6	1.390	1.540	9.7	1.573	11.6	2417	45.8	74	—	19.7	19.7	29.7	50.7
	No. 6	6.0	1.533	1.660	7.7	1.839	16.7	673	39.0	56.1	—	27.6	37.3	41.3	10.7
		7.0	1.551	1.691	8.3	1.822	14.9	485	40.7	50.6	51.9	27.6	37.3	41.8	12.1
		8.0	1.570	1.727	9.1	1.807	13.1	464	42.8	84	—	27.6	37.3	40.5	8.7
		9.0	1.588	1.734	8.4	1.791	11.3	458	41.5	96	—	27.6	37.3	41.8	11.9
		10.0	1.610	1.693	4.9	1.777	9.4	468	39.0	86	85.8	27.6	37.3	41.1	10.2
F	100% LWA	9.0	1.306	1.757	25.7	1.791	27.1	2070	47.3	33	—	19.7	19.7	33.0	67.8
		10.0	1.322	1.684	21.5	1.776	25.6	3248	49.0	56	—	19.7	19.7	35.0	78.0
		11.0	1.323	1.643	19.5	1.761	24.9	3940	46.8	65	—	19.7	19.7	34.0	—
	No. 6	6.0	1.474	1.657	11.0	1.931	23.7	706	44.3	70	—	27.6	32.9	35.6	8.3
		7.0	1.494	1.638	10.0	1.912	21.9	730	44.2	62	57.5	27.6	32.9	38.0	15.4
		8.0	1.523	1.702	10.5	1.894	19.6	489	44.8	76	—	27.6	32.9	36.5	11.0
		9.0	1.531	1.750	12.5	1.876	18.4	579	42.7	83	—	27.6	32.9	37.2	13.1
		10.0	1.550	1.665	6.9	1.859	16.6	762	41.7	141	90.1	27.6	32.9	36.8	11.9
G	100% LWA	8.0	1.286	1.655	22.3	1.722	25.3	2685	46.5	63	—	19.7	19.7	22.7	15.2
		9.0	1.331	1.592	16.4	1.709	22.1	1730	48.3	108	—	19.7	19.7	25.4	28.9
		10.0	1.303	1.548	15.8	1.696	23.2	2517	49.6	111	—	19.7	19.7	24.6	25.1
	No. 6	6.0	1.532	1.712	10.5	1.906	19.6	528	44.5	77	—	27.8	36.2	37.4	3.3
		7.0	1.539	1.462	6.3	1.888	18.5	547	45.0	63	50.7	27.8	36.2	36.9	1.8
		8.0	1.556	1.699	8.4	1.871	16.8	468	42.0	83	—	27.8	36.2	37.4	3.3
		9.0	1.592	1.665	4.4	1.853	14.1	349	44.0	84	—	27.8	36.2	38.3	5.9
		10.0	1.604	1.702	5.8	1.837	12.7	294	41.0	89	78.5	27.8	36.2	39.1	8.0

[a]Based on the Rice method (9).
[b]4-in. diameter area; pressure differential 0.25 in. water.

Packing Volume Concept for Aggregates

EGONS TONS and W. H. GOETZ, Purdue University

Aggregates with identical sieve gradation but of different characteristics, such as rounded gravel, crushed limestone, slag, and gravel, may not behave similarly in construction when used dry or mixed with a binder. The purpose of this study was to search for unifying, measurable parameters which would permit the "grading" of different aggregates so that their response in bulk to various forces would be more comparable and predictable.

A literature study and theoretical considerations suggested that the volume a particle occupies in a mass may be an important unifying concept for various kinds and sizes of aggregates. Packing volumes include solids, voids, as well as rugosity of the rock piece.

Laboratory work was performed using a certain crushed limestone, crushed gravel and rounded gravel with 4.0, 0.4, and 0.04-cc packing volumes (about $^3/_4$, $^3/_8$, and $^1/_8$-in. size). The rugosity was measured and packing volumes for different particle masses were calculated. Loose bulk volumes and volumes obtained by vibratory compaction were compared for the various rocks and sizes. The bulk volumes (densities and porosities) obtained were similar for the three aggregates indicating that the packing volume approach is a promising method for grading such aggregates on a unified basis.

•ALTHOUGH aggregates found in particle composites such as bituminous concrete, portland cement concrete and untreated granular masses are sometimes described by summation of volumes and/or surface areas, usually gradation by sieve size is employed as a primary guide in combining and designing mixes for different service conditions. It has been recognized that equal grading by sieves and batching by weights do not assure similar bulk properties when dealing with diverse types of naturally occurring (gravel) and artificially produced (crushed rock, etc.) aggregates. Yet, the difficulty in describing particles by measurable parameters of a general nature has prevailed because definitions of descriptive characteristics such as "volume," "surface area," or "surface roughness" are not as yet agreed upon for an irregular particle.

The primary purpose of this work was to search for the least number of measurable parameters for individual particles (aggregates) which would be predictive to the bulk behavior of particle composites with and without a binder (bitumen, clay, etc.).

The work reported in this paper involves selected rocks without a binder, approximately $^3/_4$ to $^1/_8$ in. in "size." The main emphasis in the laboratory work was placed on "one-size" particles, which may be represented in certain subbase, base and binder courses in highways.

LITERATURE REVIEW

A number of significant studies to characterize pieces of rock have been made. The main factors of apparent importance which have emerged are (a) particle geometry (sometimes called shape or sphericity), (b) angularity (roundness) and (c) surface roughness (texture). There are two recent and informative summaries by Gronhaug (1) and Mather (2) based on about two hundred references which discuss the various parameters. A limited number of these references are included. In the work reported here, the main emphasis was placed on a quantitative approach to describe particle geometry, volume, surface roughness, sliding friction, and packing in bulk.

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Particle Geometry

Apparently there are four primary factors affecting the shape of an aggregate particle: (a) type of rock, (b) geologic history, (c) type of crushing, and (d) sizing operation (1, 3, 4, 5, 6).

A number of qualitative terms are used to picture a piece of rock (rounded, irregular, flaky, rods, discs, blades, equidimensionals). There have been attempts to quantify particle dimensions by a so-called sphericity factor S (7, 8):

$$S = \sqrt[3]{\frac{\pi/6\, d^3}{\pi/6 \ell^3}} = \frac{d}{\ell} \tag{1}$$

Here, d is taken as the diameter of a sphere of the same volume as the rock piece and ℓ is the long dimension of the particle. Further improvement in defining the geometric shape of a particle is achieved by using three descriptive measurements: long (ℓ), short (s) and, medium (m) dimensions (9, 10, 11). For instance, in ASTM Designation C-125 (11) the ratios of ℓ/m (ℓ/w) and m/s (w/t) are adapted to classify rock pieces into four categories ranging from "flat" to "elongated." The values of the ratios for differentiating between the various classes are set arbitrarily.

The measurement of three "diameters" of a rock piece suggests the geometric form of an ellipsoid. This idea offers a great deal of flexibility. The possibility of using an ellipsoid has apparently not been much explored. Mackey (12), in connection with his work on radii of curvature measurements, uses the concept of a perfect ellipsoid and the degree of departure from this shape.

In research with particles in bulk, the effect of aggregate shape has been investigated by a number of researchers. Since the shape factor is hard to separate from other factors such as angularity, surface roughness and material properties, it is difficult to judge the true influence of particle geometry on mass density and other properties. This may account also for the apparent contradictions summarized by Gronhaug and Mather (1, 2).

Angularity or Roundness

Angularity or roundness is often described in qualitative terms such as angular, subangular, subrounded, rounded or well-rounded (13). Another more quantitative way of describing roundness is to take the ratio of the average radius of curvature (r) of the corners (n) to the radius of the maximum inscribed circle (R) of the rock piece (14, 15, 16):

$$\text{Roundness} = \frac{1}{n} \Sigma \frac{r}{R} \tag{2}$$

Since in the case of crushed angular pieces the radii of curvature are very small and difficult to measure accurately, actual angles of the sharp edges can be determined in addition to the radii of the rounded-off corners (12).

In addition to the above direct methods of measurement of angularity, experimental determinations have been made using masses of particles for relative comparisons, including refined sieving through calibrated openings, measurement of voids in bulk, measurement of angle of repose and others (1, 2). The results are varied and angularity appears to be almost always confounded with change in particle shape and roughness. Furthermore, the measurement of angularity as such cannot be used directly to calculate or predict any effects on the behavior of particles in bulk. These effects have to be determined experimentally.

Surface Roughness

The existence of surface roughness or texture of aggregate surfaces is easy to grasp but hard to measure. One way to express roughness quantitatively is by using mean surface and deviation from it (17). In a number of publications (13, 17, 18, 19, 20) on surface texture and finish, including devices and methods for measurement, there appears to be no definite agreement on classification of roughness except in qualitative terms (rough, smooth, furrowed, grooved, scratched, ridged, pitted, dented, striated, frosted, etched, etc.). Measurement of actual surface area of a certain polished and rough limestone has shown that the rough area was about three times greater than the polished area (21). Blanks (22) has pointed out that there are two kinds of surface roughness: abrupt and undulatory.

A simple quantitative method for measuring surface roughness for smooth, level surfaces has been proposed by Bikerman (26). He coated flat sawed rock plates with asphalt, scraped the excess down to the stone and used the amount of asphalt left as an indicator of surface roughness (and absorption).

Angularity and Roughness Combined

At least intuitively, the shape or geometry of an aggregate piece is a separate parameter. There is a question, however, whether angularity and roughness do not overlap, especially in the case of crushed rock. Gronhaug (1) proposes to combine angularity and texture (roughness) into one term: form. Another, possibly unifying, term would be rugosity (17). Here the adjective "rugged," which stands for rough, uneven, jagged, ridged, or wrinkled, seems to be applicable to irregular particles of aggregates. (It may also encompass some of the surface voids of the rock.)

Sliding Friction and Compaction

If all aggregate pieces were ideal, smooth, one-size spheres and they were packed in a simple cubical arrangement, the voids (porosity n) in the mass would be about 47.6 percent; in the densest tetrahedral packing this value would be reduced to 26.0 percent. For randomly packed spheres and irregular particles, the porosities usually vary between these two extremes.

During packing of rocks, either by gravity-flow or by some mode of densification such as vibration, some relative movement between pieces takes place. The actual contact area between two pieces (rough or smooth) is small and in the order of about 0.01 percent of the apparent contact area (23). For two hard rock pieces:

$$F = W\frac{s}{y} \tag{3}$$

where

- F = force to drag one particle along the surface of another,
- W = load on the particle (contact),
- s = shear resistance, and
- y = yield value.

The ratio s/y should be nearly independent of the nature of the rock itself, since s and y tend to vary together (23). Thus for rocks with clean surfaces, the force F should be dependent on the load only. In other words, two masses of different one-size rock pieces subjected to identical load W (compaction) should respond in a similar manner.

From physics (27), the velocity in free fall neglecting air friction is

$$V = \sqrt{2gx} \tag{4}$$

where

- V = velocity of particle,
- g = gravitational constant, and
- x = distance of fall.

Therefore, if two masses of particles are "poured" from identical heights into a container, their velocities will be about the same regardless of the individual particle mass.

In vibratory compaction (sinusoidal) D'Appolonia (25) suggests that the peak acceleration in g's is important and, in order to get noticeable compaction, acceleration over 1 g is necessary. The useful equation for relating frequency and amplitude to acceleration is

$$f^2 = \frac{a_g}{0.102A} \tag{5}$$

where

f = frequency, cps;
a_g = acceleration, in g's; and
A = peak amplitude, in.

Summary of Literature

Only selected groups of references, which are concerned primarily with concepts relative to measurement of particle properties and behavior, have been recalled. No attempt was made to go into a discussion of published data based on experimental evidence involving several factors at the same time. The survey of the literature has made the following apparent:

1. There is no unified agreement on the parameters of importance for quantitative characterization of rock particles.
2. There is a need to find and tie in such characteristics to the behavior of particles with different compositions and sizes in bulk (bulk densities, flow characteristics, etc.).

THEORETICAL CONSIDERATIONS

This section describes the background reasoning leading to laboratory measurements and particle packing volume as a proposed useful concept when dealing with an aggregate mass. The main emphasis is placed on monovolume (one-size) particles, about 4, 0.4 and 0.04 cc in volume (¾, ⅜ and ⅛-in. "size").

Particle Volume—Ellipsoid Geometry

The bulk volume of a number of particles in a container is, among other things, a function of the volumes of each of them. It is assumed that the volume which a particle occupies in a mass of other particles largely determines the density and the voids in bulk and that therefore this volume is important as far as the response of the composite to various forces is concerned. The problem at hand is to attempt to define the volume of a particle, especially if it is irregular in shape as well as rough (high rugosity).

To define the volume of any particle, it is convenient to have a geometric form which lends itself to numerical description and analytical manipulation. As pointed out, the measurement of long, medium and short dimensions of a particle is not a new idea. Since in the field of aggregates there are practically no cubes, spheres, rods or other regular shapes, why not try to fit an ellipsoid for all types of particles as a geometric form?

The volume of an ellipsoid is simply

$$V = \frac{\pi}{6} \ell ms = 0.524 \ell ms \tag{6}$$

The equation for surface area is more complicated and a prolate spheroid is often used as an approximation.

Packing Volume of a Particle

All surfaces of particles possess some kind of roughness. The peaks or asperities of the roughness are spaced randomly. For example, if two pieces of crushed limestone are in contact with each other, the peaks and the valleys will not be able to mesh

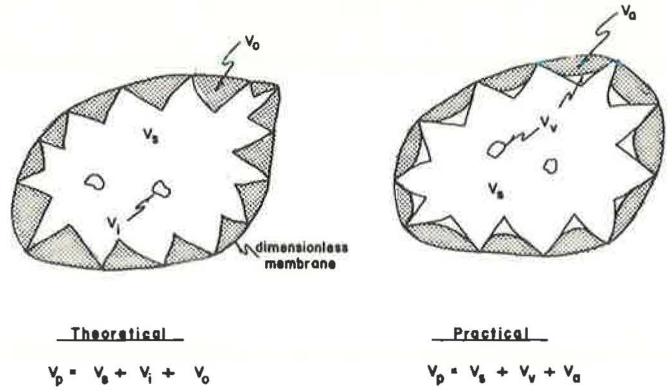

Figure 1. Components of a particle packing volume.

like two carefully cut gears. Instead, the particles will touch one another at the high spots, and only a small portion of the areas will be in contact (23). Therefore, the volume which a piece of rock occupies in a mass of other particles encompasses not only the volume of solids and internal voids, but also the volume of the dips and valleys of the particle surface which may be called "outside voids" (Fig. 1a). These outside voids are primarily a function of the rugosity of a surface. The term "packing volume" when applied to a particle as used in this study is that volume which the particle occupies in a mass of particles or

$$V_p = V_s + V_i + V_o \tag{7}$$

where

V_p = packing volume,
V_s = volume of solids of the particle,
V_i = volume of internal voids, and
V_o = volume of outside voids or surface irregularities.

The packing volume can be pictured as a volume enclosed by a dimensionless, flexible membrane stretched along the surface of a rock (Fig. 1a).

In the laboratory, packing volume can be measured by immersing the rock in asphalt, removing the excess asphalt down to the peaks of the surface and weighing the piece in air and water (Fig. 1b):

$$V_p = \frac{W_t - W_w}{G_w} \tag{8}$$

where

W_t = total weight, rock plus asphalt, in air;
W_w = weight in water; and
G_w = unit weight of water.

The weight of a rock piece to give a certain desired packing volume for practical application can be derived as follows (Fig. 1b):

$$V_p = V_s + V_v + V_a \tag{9}$$

$$V_p = \frac{W}{G_{s+v}} + V_a \tag{10}$$

$$W = G_{s+v} (V_p - V_a) \tag{11}$$

where

V_v = internal and surface voids unfilled with asphalt,
V_a = volume of asphalt after scraping,
W = weight of dry rock piece,
G_{s+v} = specific gravity of solids plus voids including those under the asphalt coating,
V_p = packing volume, and
V_s = volume of solids of the particle.

The volume of asphalt, V_a, will depend on the surface area A and surface roughness R of the rock piece. Therefore, Eq. 11 can be rewritten

$$W = G_{s+v} (V_p - AR) \tag{12}$$

where $R = V_a/A$ or the volume of asphalt on the rock piece after scraping, divided by the smooth "membrane" surface area of the rock. The equation for G_{s+v} is

$$G_{s+v} = \frac{W}{\frac{W_t - W_w}{G_w} - \frac{W_a}{G_a}} \tag{13}$$

where

W = weight of the rock piece (or pieces),
W_t = weight of the rock piece + asphalt in air,
W_w = weight of the rock piece + asphalt in water,
W_a = weight of asphalt,
G_a = specific gravity of asphalt, and
G_w = unit weight of water.

The value G_{s+v} is constant for a given aggregate piece provided a certain procedure is followed just as in any test for specific gravity of aggregates. However, if two laboratories use two different methods and obtain two different G_{s+v} values, Eq. 12 still holds, since rugosity R changes in unison with G_{s+v} (Fig. 1b). The knowledge of G_{s+v} may be useful for obtaining rugosity R factors without resorting to scraping.

As mentioned in the literature review, for a given aggregate the surface rugosity R is higher for larger pieces as compared to smaller ones. During crushing operations, cracks propagate along the path of least resistance, leaving fine surface roughness superimposed on longer undulating roughness. The smaller the rock, the less the inclusion of larger undulations and the smaller should be the rugosity factor R.

Grading by Sieves and Packing Volume

Grading by sieves alone does not assure good control of particle packing volume. The volume of an ellipsoid is $V = \pi/6 \; \ell ms$ and the medium and short dimensions are primarily responsible for passage through a square-hole sieve. If the sieve hole size is H, it can be shown that the volume V of the rock passing it is

$$V = \ell \left(\frac{\pi}{6} s \sqrt{2H^2 - s^2}\right) \tag{14}$$

If s and H are fixed, this volume is $V = \ell K$, where K is a constant. This shows quantitatively that aggregates passing a given sieve (and having identical m and s values) will have uncontrolled volumes directly proportional to the length ℓ of the rock piece (Fig. 2). Thus the particle volume distributions should be different for different rocks of identical sieve size. However, for a given aggregate, it should be possible to use sieve grading to predict volume grading through correlation factors.

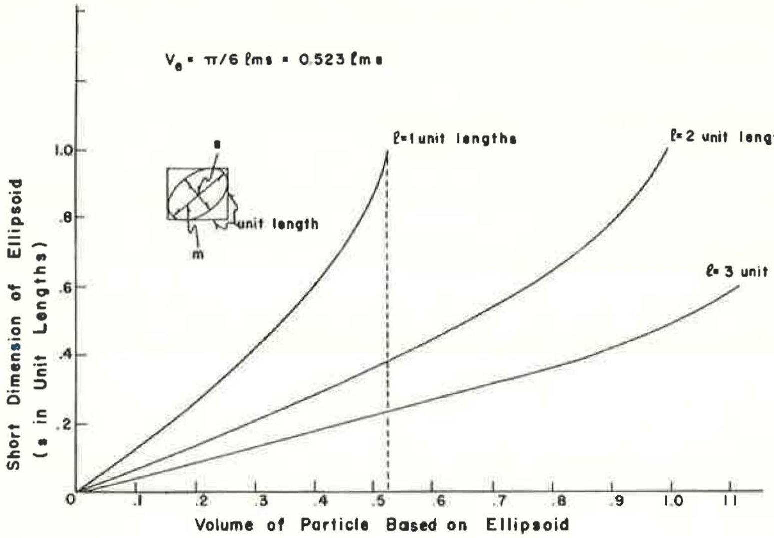

Figure 2. Theoretical volumes for ellipsoids passing a square sieve.

Packing Densities of Perfect Ellipsoids

Packing densities and voids (porosity) for perfect spheres under certain configurations have often been used for comparisons in particle studies. The volume of a sphere is $V_S = \frac{\pi}{6} d^3$, of an ellipsoid, $V_e = \frac{\pi}{6} \ell ms$. For the case of one-size smooth spheres in simple cubical packing, the porosity n = 47.6 percent. If, instead, ellipsoids of identical ℓ, m, and s values are packed in a similar manner:

$$n = 1 - \frac{\pi}{6} \ell ms \left(\frac{1}{\ell} \times \frac{1}{m} \times \frac{1}{s}\right) = 0.476 \ (47.6\%)$$

For spheres in cubic-tetrahedral packing, n = 39.5 percent, and for ellipsoids in similar arrangement,

$$n = 1 - \frac{\pi}{6} \ell ms \left(\frac{2}{\ell\sqrt{3}} \times \frac{1}{m} \times \frac{1}{s}\right) = 0.395 \ (39.5\%)$$

Finally, in the densest tetrahedral packing for spheres n = 26.0 percent. The packing for ellipsoids in similar:

$$n = 1 - \frac{\pi}{6} \ell ms \left(\frac{2}{\ell\sqrt{2}} \times \frac{1}{m} \times \frac{1}{s}\right) = 0.260 \ (26.0\%)$$

From these calculations it is apparent that dense, loose, and intermediate packing of perfect equidimensional ellipsoids give voids (porosities) identical to those between ideally packed spheres.

So far, consideration has been given to the shape (ellipsoid) and rugosity (surface roughness) of the particle. It should be pointed out that rugosity as measured by scraping may also be influenced by angularity (the more angular the rock, the higher the rugosity). Sharp corners of rock, however, are not accounted for in the packing volume concept.

Bulk Volumes

Calculations have shown that volumes of voids for one-diameter spheres and equidimensional ellipsoids are identical under ideal packing conditions. Since the ellipsoid gives a good approximation for the geometry of irregular particles, it is conceivable that the particles will also pack similarly to spherical particles in a composite.

The void content (or porosity) of a mass of small or large one-volume particles should be the same, regardless of the type of rock and shape of particle, just as it is with ideal spheres. Thus the ratio of the number of small particles N_1 to the number of large ones, N_2, should be indirectly proportional to their packing volumes V_1 and V_2:

$$\frac{N_1}{N_2} = \frac{V_2}{V_1} \tag{15a}$$

or

$$N_1 V_1 = N_2 V_2 \tag{15b}$$

When packing or compacting different kinds of particles, identical procedures are necessary to obtain comparable results. Thus, for example, when a mass of rocks is "poured" into a given mold or container, the rocks must be deposited from a similar height and within an identical time interval (12). If vibratory compaction is used, the peak acceleration must be the same (25).

Finally, the porosity, or voids, as considered here, is not the absolute porosity of the bulk since the basis of the "solid" volume is packing volume, which includes surface roughness or surface voids. Thus, one-volume rounded gravel and crushed stone may have identical packing porosities in a mass, but the amount of liquid such as water or mercury to fill the aggregate voids would be greater in the case of the crushed limestone.

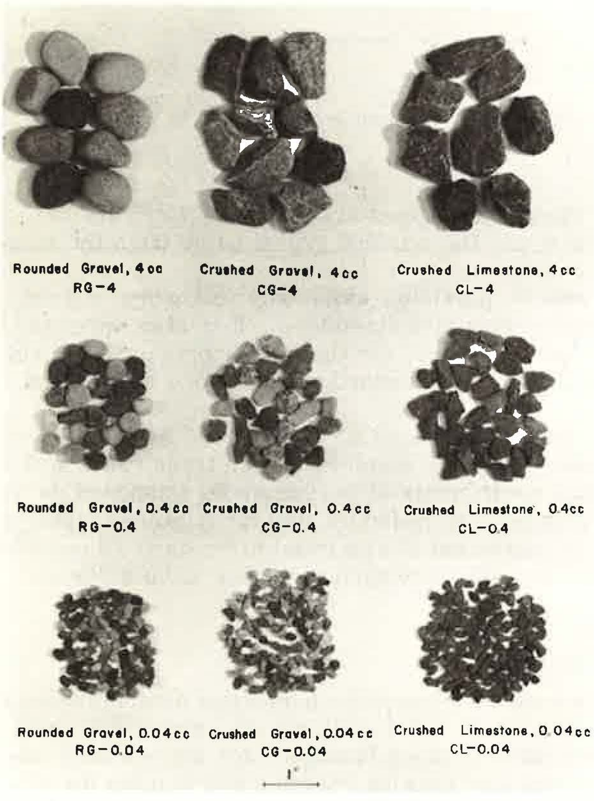

Figure 3. Types and sizes of rocks.

EXPERIMENTAL WORK

The reasoning developed by theoretical considerations was tested in the laboratory using three types of rocks (crushed limestone, crushed gravel, and rounded gravel) with three distinct packing volumes about one decade apart (4, 0.4, and 0.04 cc). In terms of "sizes," the rocks were about ¾, ⅜, and ⅛ in., respectively (Fig. 3). In addition, comparative measurements were made using ½-in. smooth glass spheres (marbles). The surface rugosity and geometric shape were measured, packing volumes were calculated and weights for identical bulk volumes were predicted for the various rocks and sizes. Loose bulk volumes and volumes after vibratory compaction were measured and compared to check the validity of the packing volume concept.

Description of Aggregates

The three aggregates were selected on the basis of differences in rugosity (crushed vs rounded) and

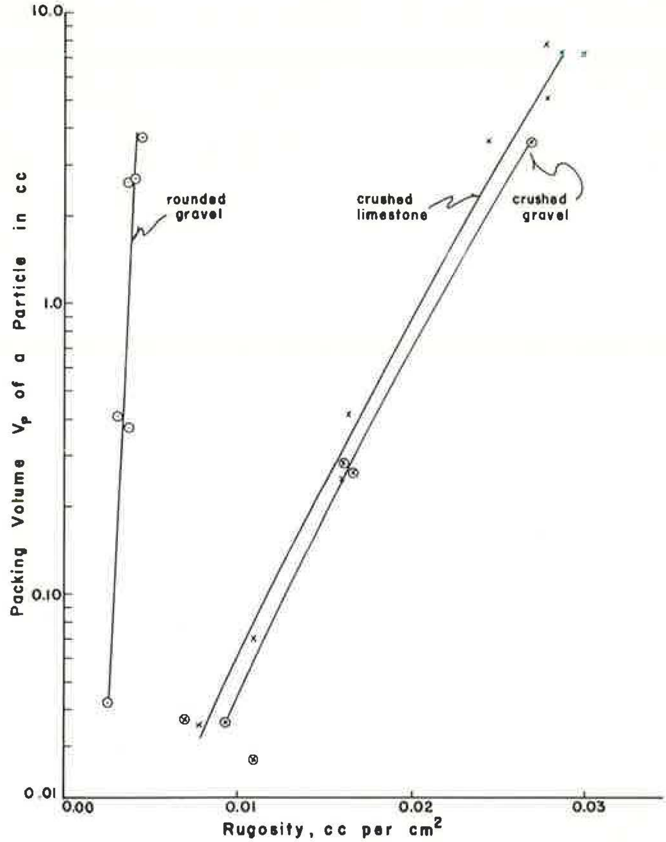

Figure 4. Rugosity vs packing volume for the three types of rocks.

composition (sedimentary vs mixed). These three types are also frequently used in highway construction. The crushed gravel and the rounded gravel came from the same source.

It is apparent that one-size or one-volume particles exist only in theory. Even smooth, one-size glass spheres do not have identical diameters. It is also impossible to produce one-volume rock particles, and therefore, the three categories of rock volumes are actually mean volumes with a controlled standard deviation and about equal coefficient of deviation.

The 0.04-cc rocks were obtained by dividing the fraction between the No. 4 and No. 6 sieves into portions retained on the No. 5 and No. 6 sieves. Then these two fractions were combined accordingly to get similar coefficients of deviation, D, (standard deviation divided by the average weight of particle, multiplied by 100) for the three types of rocks, based on weights of particles. A convenient D was found to be about 15 percent. Similar handling of the 0.4 and 4-cc rocks, using appropriate sieves, gave a desired D = 15 percent in each case.

Measurement of Rugosity and Packing Volume

Packing volume of particles can be measured without the numerical determination of rugosity (Fig. 1b). It is convenient, however, to have available the characteristic relationship between rugosity and different particle sizes (volumes) for a given rock because it provides a basis for calculating particle packing volumes and weights for other "sizes" than those used in the actual determination (Eqs. 10 and 12). Rugosity value is

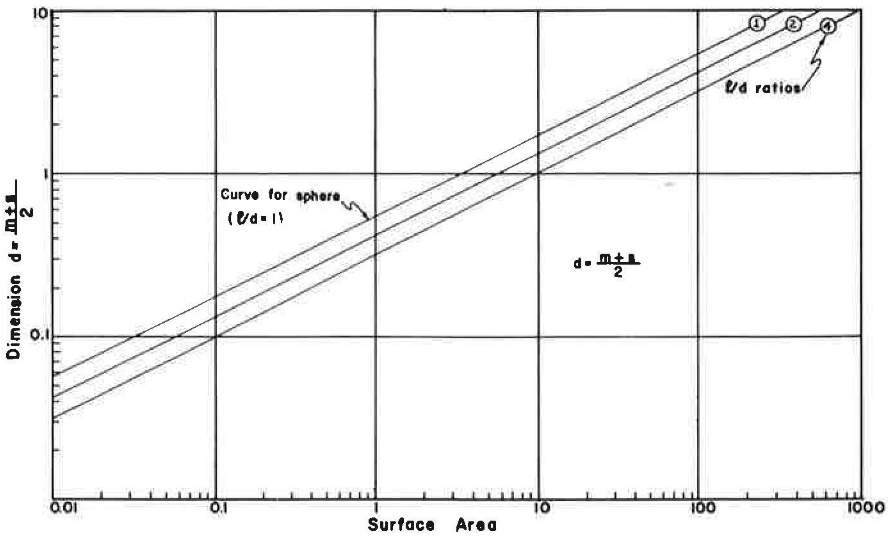

Figure 5. Graph for determining surface area for a prolate spheroid.

also needed when calculating the amount of a binder, such as asphalt, to be mixed with aggregate. The primary reason for measuring rugosity here is to show that it adds to the particle volume in bulk.

Figure 4 shows the rugosity as it changes with the volume of each rock. To obtain the curves, rock pieces were drawn at random from a mass of other pieces of the same size and then washed, dried, weighed, and heated to 300 F in a compartmentalized container. They were then covered by a 55 penetration asphalt at 300 F for 30 min after which each coated rock was dipped in ice water. The excess asphalt was scraped off each piece, down to the peaks of roughness.

The scraping was done with a razor blade, applying its straight edge and avoiding use of the corners. This operation was tedious and required some patience and skill. After scraping, crushed rock and rounded gravel look very much alike except for some sharp angles of the former. The particles were weighed again in air and in water, giving direct measurement of their packing volumes. Furthermore, the three dimensions ℓ, m and s for each rock piece were measured and their "membrane" surface area was calculated using the simplified equation of prolate spheroids:

$$A = \frac{1}{2} \pi d \left(d - \frac{\ell}{K} \sin^{-1} K\right)$$

where

A = surface area of particle ("membrane" area),

$d = \frac{m+s}{2}$, and

$K = \frac{(\ell^2 - d^2)^{1/2}}{\ell}$.

In practice the areas for each rock piece were obtained using a graph identical to Figure 5 but on an expanded scale. Using the preceding data, rugosity values were calculated for different sizes and kinds of rocks (Fig. 4):

$$\text{Rugosity} = \frac{\text{Amount of asphalt on rock, cm}^3}{\text{Surface area of rock, cm}^2}$$

The manually measured m, ℓ, and s values used in surface area calculations for each rock were also useful for calculating packing volumes and comparing them with

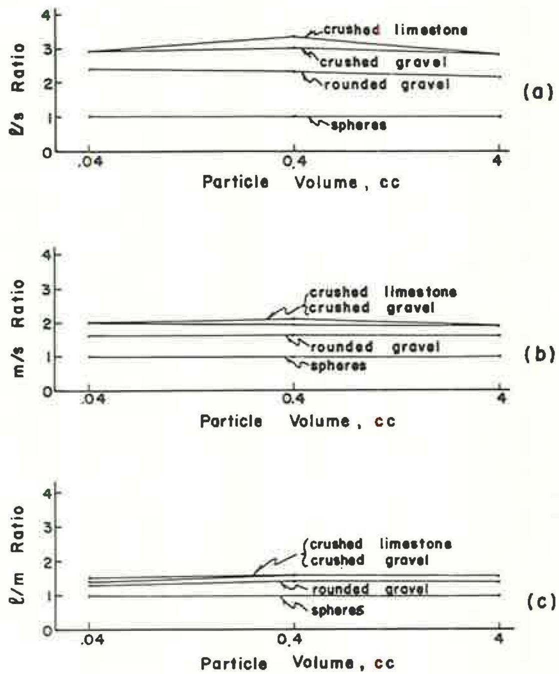

Figure 6. Ratios for various aggregates.

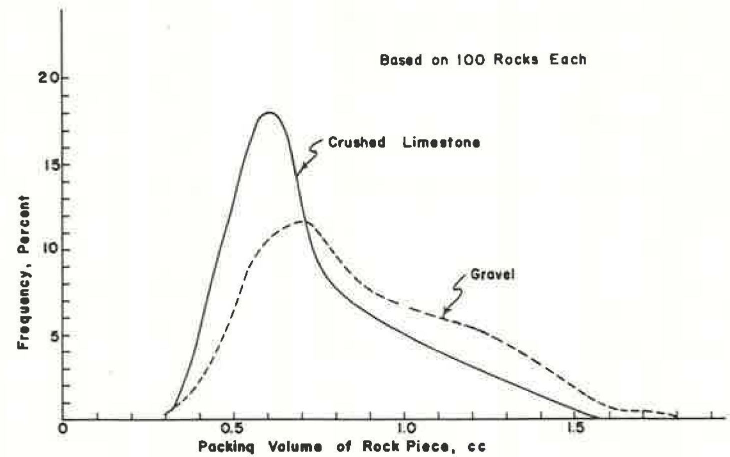

Figure 7. Example of particle volume distribution for $\frac{1}{2}$ to $\frac{3}{8}$-in. crushed limestone and partially crushed (about 50 percent particles) gravel.

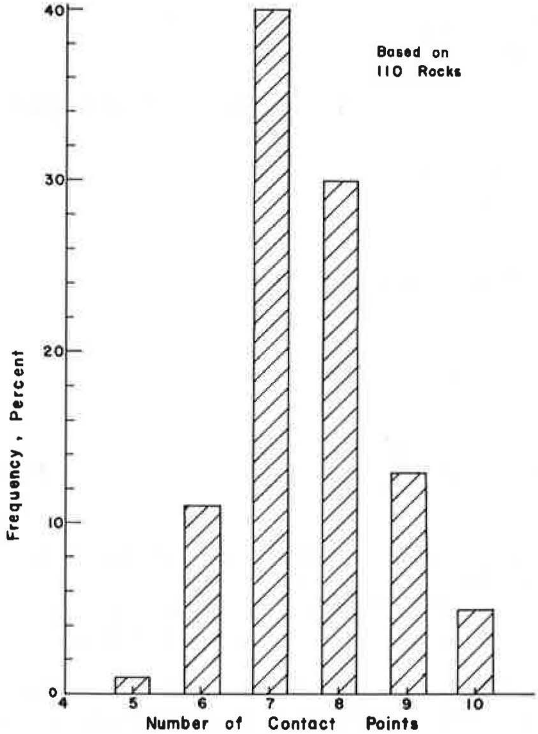

Figure 8. Example of contact point distribution, one-volume rocks.

those measured by the water displacement method. Statistical difference analysis on 90 rocks by the two methods indicated that direct measurement using the assumed shape of ellipsoid is in good agreement with the results of the volume-by-water-displacement method (for detailed analysis see Appendix A). This suggests another method for measuring packing volumes of particles.

To determine differences in the shape of ellipsoids, comparisons were also made among ℓ/s, ℓ/m, and m/s ratios for various fractions of the same rock (Fig. 6). The curves indicate a slight tendency for the 0.4-cc rocks to have higher ratios compared to the smaller and larger rocks of the same kind. Each point (Fig. 6) is an average of 10 measurements. A numerical analysis in Appendix B indicates that the differences do not appear important.

Sieve Size and Particle Volume

The particle volume distribution of crushed limestone and rounded gravel (or any two aggregates) is expected to be different even if taken from the same sieve size fraction. Figure 7 shows an example of packing volume distribution curves obtained for a certain ½ to ⅜-in. crushed limestone and the same size gravel. In the case of the limestone, there is a tendency for the average volume of the particles to be smaller. It is expected that each type of rock from a given quarry (effect of crushers assumed constant), if sieved by a given procedure, will have a characteristic particle volume distribution on each sieve. Once this distribution is known, aggregates could be combined on the basis of particle packing volume distribution, using sieve grading and proportioning according to a "packing volume formula."

Number of Contact Points

The number of contact points in a simple cubical packing and in a tetrahedral (dense) packing of spheres is 6 and 12, respectively. The same number applies to ellipsoids in similar loose and dense packings. The number of contact points for the nine groups of one-volume aggregates were determined at one particular mass density, using disturbance of the asphalt coating for detection. The procedure involved mixing about 500 cc (½ qt) of rocks at 300 F with just enough 55 penetration asphalt to fill the volume of rugosity. The coated rocks were then placed in a container so as to obtain about equal porosities. Then the mass was cooled to 0 F, the rocks separated and the contact points counted. An example of a distribution curve is shown in Figure 8. Again, no significant differences in number of contacts between the kinds and sizes of rocks were found (see Appendix C). It is expected that the number of contact points increases with increased compaction of the particles (24).

Identical Bulk Volumes with Identical ΣV_p of Rocks

As given by Eq. 10, the packing volume V_p of each individual rock can be calculated:

$$V_p = \frac{W}{G_{s+v}} + AR$$

The number N of particles needed to give a certain bulk packing volume ΣV_p of a mass of particles is

$$N = \frac{\Sigma V_p}{V_p}$$

The total weight ΣW of such a mass of N particles would be

$$\Sigma W = NW = \frac{\Sigma V_p}{V_p} W$$

or

$$\Sigma W = G_{s+v} (V_p - AR) \frac{\Sigma V_p}{V_p} \tag{16}$$

Eq. 16 permits the calculation of how much by weight of a certain "size" of rock is to be taken to obtain a given packing volume ΣV_p for a mass of particles.

If Eq. 16 holds and if sliding friction is similar with the three rocks tested, they should have similar bulk volumes for identical ΣV_p. This theory was tested by two methods: free fall and vibratory compaction.

For the free-fall test, the three rocks and three sizes (nine batches altogether) were prepared with identical total packing volumes of 800 cc. The dry, clean rocks were then poured into a cylindrical container 5 in. in diameter and 5 in. high from an average height of 3 in. and within a time interval of 10 sec. The resulting bulk volumes were then determined for each of the rock types and sizes. Graphical comparisons (Fig. 9) and statistical analysis showed that there was no significant difference in the bulk volumes thus obtained.

In another series of tests, sinusoidal vibration was applied to the various rocks in bulk with peak acceleration of 1.5 times gravity and without surcharge. The frequency

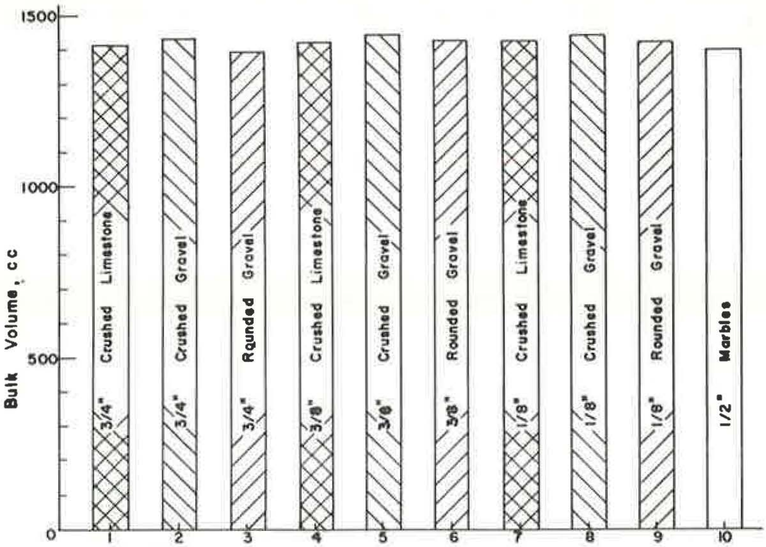

Figure 9. Loose volumes of ΣV_p = 800 cc, 3 in. drop, 10 sec.

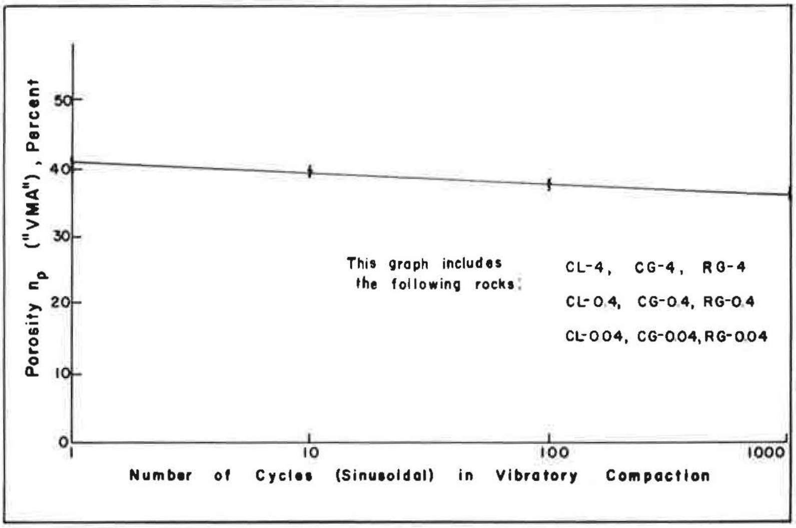

Figure 10. Porosity or voids in the bulk after various numbers of cycles at 1.5 g's peak acceleration (all rocks and sizes are included).

chosen was 20 cps and the bulk-mass volumes of the rocks were measured at 1, 10, 100, and 1000 cycles of vibration. Since the same rock samples had to be used repeatedly in these test series, the compaction was not extended beyond 1000 cycles to minimize particle degradation. The results are summarized in Figure 10 and the data tabulation is given in Appendix D. It was convenient here to compare porosities n_p instead of bulk volumes.

The measurements again showed that all rocks had similar densification trends. The bulk volumes and porosities n_p obtained at each of the indicated cycles were similar for a given sum of packing volumes (ΣV_p) regardless of rock type or size; this was expected from the theoretical considerations.

As an additional check, ½-in. marbles with R = 0 and the same ΣV_p were included in both of the above "compaction" tests. Marbles behaved quite similarly to the various one-size rocks.

DISCUSSION OF RESULTS

The primary goal of this study was to search for a unifying approach to the characterization of aggregate particles of various kinds and sizes. The scope has been limited to three types and three sizes of aggregates in the coarse aggregate range.

The central hypothesis was that the volume which a particle, large or small, angular or rounded, smooth or rough, occupies in a mass of other particles is an important characterizing factor as far as bulk properties under a defined "compaction" are concerned. The test results show that a particle packing volume concept is useful in defining the characteristic space a piece of rock occupies in a bulk. This packing volume can be obtained using Eq. 10, in which G_{s+v} is defined by Eq. 13.

However, when dealing with bulk density (or porosity) another type of specific gravity for a rock piece based on packing volume V_p may be useful. This could be called packing specific gravity, G_p. If W is the dry weight of a rock piece and V_p is its packing volume, then

$$G_p = \frac{W}{V_p} \tag{17}$$

Numerically, G_p would be the lowest of all commonly used specific gravity values since the volume includes surface voids. It would be constant for one given volume of rock, but would vary with rock size and type because surface roughness and surface area, which are functions of rock size and type, determine surface voids. From Eq. 17, a weight-volume relationship for a number of particles taken together is

$$\Sigma W = \Sigma V_p G_p \qquad (18)$$

If a given total packing volume ΣV_p (say, $\Sigma V_p = 1000$ cc) of any of the nine individual rock-size groups were desired and designated as

$$\Sigma V_{p1}, \Sigma V_{p2}, \ldots, \Sigma V_{p9}$$

then the total weight ΣW needed to give these constant volumes could be calculated using Eq. 18. It is apparent that generally

$$\Sigma W_1 \neq \Sigma W_2 \neq \ldots \neq \Sigma W_9$$

However, if identical free fall or vibratory "compaction" is employed to densify the above nine batches of rock, the bulk volumes V_B will be equal or

$$V_{B1} = V_{B2} = \ldots = V_{B9}$$

This also leads to the conclusion that weight per unit volume of these particles will be different depending on G_p and the type of compaction. The packing volume concept at this stage of development permits the calculation of "one-size" rock weights which will produce identical bulk volumes under identical compaction. This concept is also expected to permit the prediction of flow of an aggregate mass mixed with a binder and subjected to load.

Thus, laboratory findings support the general line of theoretical considerations discussed previously. Angularity did not prove to be a distinctive feature, even though some of this is taken care of by the rugosity factor R. Also, shape did not have a noticeable influence. It is difficult to say much about the effects of ℓ/s ratios larger than 3.4, since the rocks used did not exceed this value.

In this study, the particle volume distributions were rather narrow, or practically one-volume. It is apparent that mixed-volume particles will have more complex packing behavior. Differences will be introduced by the variations in rugosity R with particle volume and the relative amounts of each volume fraction. However, it is expected that the concepts of packing volume can be extended to the case of multi-size particles probably down to the filler size. This still remains to be demonstrated.

The immediate practical goals behind this research included consideration of aggregates "grading" close to those used in "one-size" and open-graded mixes, such as is sometimes the case in subbase, base and possibly binder courses for highways. The following translations into practice are pertinent:

1. One-size crushed rock and one-size rounded gravel of identical sieve sizes are not identical when graded by particle volume. Response to compaction and service performance is expected to be different.
2. If different aggregates were graded by particle volume, vibratory compaction in thick layers should require identical effort and their performance would not be expected to be greatly different provided other conditions were similar.
3. If a binder, such as asphalt or clay, is added, rugosity as well as lubricating binder has to be considered in obtaining identical workability.
4. The packing volume concept appears to offer promise as a unified mix design approach for diverse types of aggregates used in construction and research studies.
5. Grading by sieve size is still expected to be useful for proportioning aggregates by particle volume.

CONCLUSIONS

These conclusions are based on laboratory measurements and analysis of certain crushed limestone, crushed gravel and rounded gravel aggregates of three particle volumes: approximately 4, 0.4, and 0.04 cc (about ¾, ⅜, and ⅛ in., respectively). Although several important aggregate variables have been included on a fairly broad scale and it is probable that these conclusions can be applied to a wider range of aggregates than those studied, extension of the validity of the findings beyond the specific scope of this study remains to be demonstrated.

1. Particle packing volume, the volume which a piece of aggregate occupies in a mass of particles, is a parameter unifying the bulk behavior of the coarse aggregates tested.
2. The packing volume can be quantitatively defined by (a) particle geometry and "surface" area, and (b) rugosity of the particle.
3. Rugosity includes primarily surface roughness, some accessible pores, plus some angularity of a particle. It is directly proportional to the volume of a given rock type.
4. Geometry of irregular particles can be satisfactorily characterized by an ellipsoid.
5. One-volume ellipsoids ideally pack like spheres, giving porosities identical to those of spheres, regardless of size and dimensions.
6. The three rocks, with three sizes each, have identical porosities with identical total packing volumes ΣV_p when (a) poured into a container under identical conditions, or (b) compacted by vibratory compaction under identical conditions.

ACKNOWLEDGMENTS

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Appendix A

COMPARISON OF MEASURED ELLIPSOID
VOLUMES WITH VOLUMES BY WATER DISPLACEMENT METHOD

Numbers are percentages of difference between the two methods

Rock → / Volume ↓	CL	CG	RG	Ti.	n	X_{ij}^2
	+ 10.8	+ 2.0	+ 5.3			
	- 16.8	- 6.1	- 8.3			
	+ 9.9	+ 1.3	+ 12.6			
	+ 0.3	- 11.2	- 7.4			764.1
4	- 8.3	- 4.4	+ 10.4	10.1	30	722.0
	- 8.7	+ 2.1	- 11.7			822.7
	- 1.0	- 8.6	+ 9.4			
	+ 2.8	+ 17.9	- 12.7			2308.8
	+ 5.4	+ 11.6	- 0.8			
	+ 9.2	+ 1.0	+ 4.1			
	- 3.2	- 12.1	+ 7.0			
	- 7.1	+ 8.9	- 5.8			
	+ 7.9	+ 10.5	- 11.9			
	- 1.4	+ 6.2	- 0.3			
	+ 1.3	- 2.9	- 6.3			741.2
0.4	- 11.8	- 2.6	+ 13.4	7.7	30	575.9
	- 6.8	+ 7.6	+ 3.8			640.2
	+ 1.4	- 3.6	+ 6.5			
	+ 4.8	- 10.7	- 10.9			1957.3
	+ 20.1	+ 1.1	+ 4.6			
	+ 9.9	- 8.0	+ 3.5			
	- 18.3	+ 2.2	- 2.8			
	+ 7.6	+ 4.6	+ 16.2			
	- 12.1	- 7.2	+ 3.8			
	- 1.3	+ 11.4	- 6.2			767.6
0.04	+ 3.1	+ 3.2	- 6.5	- 2.4	30	329.2
	- 1.3	- 2.0	- 7.1			601.0
	+ 2.6	+ 2.8	- 6.1			
	+ 0.6	- 1.3	- 7.4			1697.8
	+ 10.5	- 5.8	+ 9.0			
T.j	10.1	7.9	- 2.6	T.. = 15.4		
n	30	30	30	N = 90		
ΣX_{ij}^2	764.1	722.0	822.7			
	741.2	575.9	640.2			
	767.4	329.2	601.0			
	2272.7	1627.1	2063.9	$\Sigma\Sigma X_{ij}^2$ = 5963.7		

Volume Difference Analysis for CL-4 Stone

(Correlated Samples)

Model: $d_i = \overline{D} + \epsilon$

H: $\overline{D} = 0$

$D = 0.36$

$$S_{\bar{d}} = \sqrt{\frac{S^2}{10}}$$

$$S^2 = \frac{764.1 - \frac{(3.6)^2}{10}}{9} = 84.4$$

$$S_{\bar{d}} = \sqrt{\frac{84.8}{10}} = 2.9$$

$$t = \frac{\bar{d}}{S_{\bar{d}}} = \frac{0.36}{2.9} = 0.12$$

$t_{9\,(5)} = 2.26 \qquad 0.12 < 2.26$ Hypothesis accepted

Similar results were obtained for:

	CG-4	RG-4
CL-0.4	CG-0.4	RG-0.4
CL-0.04	CG-0.04	RG-0.04

In addition, volume difference analyses were performed for:

(1) CL-4, CL-0.4, CL-0.04 combined
(2) CG-4, CG-0.4, CG-0.04 combined
(3) RG-4, RG-0.4, RG-0.04 combined

(4) CL-4, CG-4, RG-4 combined
(5) CL-0.4, CG-0.4, RG-0.4 combined
(6) CL-0.04, CG-0.04, RG-0.04 combined

(7) All nine combinations

No significant differences were found.

Appendix B

ANALYSIS OF ℓ/s RATIOS

Hypothesis: Average ℓ/s ratios for CL are equal (numbers are actual ratios)

Volume of Rock, cc

	4	0.4	0.04	
	3.3	1.9	2.5	
	2.0	4.3	3.2	
	2.1	4.3	2.5	
	2.7	3.5	3.4	
	3.1	3.7	2.5	
	2.5	3.7	3.5	
	2.6	2.4	4.4	
	2.0	3.7	3.6	
	2.9	3.4	2.1	
	1.9	5.5	2.1	
$T._j$	25.1	36.4	29.8	$T.. = 91.3$
n	10	10	10	$N = 30$
ΣX_{ij}^2	65.2	141.5	93.9	$\Sigma\Sigma X_{ij}^2 = 300.6$

$$SS_{total} = \Sigma (X_{ij})^2 - \frac{T..^2}{N} = 300.6 - \frac{(91.3)^2}{30} = 22.8$$

$$SS_{treat} = \Sigma (T._j)^2/n - \frac{(T..)^2}{N} = \frac{(25.1)^2 + (36.4)^2 + (29.8)^2}{10} - \frac{(91.3)^2}{30} = 6.5$$

$$SS_{err.} = SS_{tot.} - SS_{treat.} = 22.8 - 6.5 = 16.3$$

$F_{2,27} = 5.40$

$F_{2,27(0.99)} = 5.45$

$5.40 < 5.45$

Hypothesis accepted

Randomly chosen Crushed Gravel (CG) and Rounded Gravel (RG) pieces had the following ℓ/s ratios:

Crushed Gravel

Volume of Rock, cc

4	0.4	0.04
2.8	3.0	1.3
1.9	2.9	3.6
5.6	5.3	1.7
2.3	3.8	3.3
1.8	2.3	4.0
1.8	1.8	2.0
1.8	5.7	2.0
2.5	2.8	1.7
3.3	2.8	2.0
2.7	4.0	2.8

Rounded Gravel

Volume of Rock, cc

4	0.4	0.04
1.9	3.5	1.7
3.6	1.4	1.8
1.7	2.6	1.7
1.6	2.4	2.8
3.2	1.7	2.6
1.3	2.6	1.9
1.9	2.3	1.7
1.9	2.2	1.4
2.1	2.1	2.0
2.1	2.6	2.0

For the sake of brevity ℓ/m and m/s ratios are given only in graphical form, Figure 6. Detailed data available from the authors.

Appendix C

ANALYSIS FOR NUMBER OF CONTACT POINTS

Hypothesis: Average number of contact points are equal in the following rocks:

(Number of contact points; each number is an average for 10 rocks)

	CL-4	CL-0.04	RG-4	RG-0.04	Marbles	
	7.6	7.4	7.6	7.0	7.6	
	8.0	7.8	7.2	7.8	8.2	
	7.2	8.6	7.4	7.6	7.2	
	7.6	7.4	7.2	7.6	8.0	

Coded $X' = \frac{X-7.6}{0.1}$ $\quad \bar{X} = 7.6$

	CL-4	CL-0.04	RG-4	RG-0.04	Marbles	
	0	-2	0	-6	0	
	4	+2	+4	+2	+6	
	-4	+10	-2	0	-4	
	0	-2	-4	0	+4	
$T_{.j}$	0	+8	-2	-4	+6	$T_{..} = +8$
n	4	4	4	4	4	$N = 20$
ΣX_{ij}	32	112	36	40	68	$\Sigma\Sigma X_{ij}^2 = 288$

$SS_{total} = 288 - \frac{64}{20} = 285$

$SS_{treat} = \frac{120}{4} - 3 = 27$

$SS_{err} = 261$

$F_{4, 15} = 0.39$

$F_{4, 15\,(0.99)} = 4.89$

$0.39 < 4.89$ Hypothesis accepted

Appendix D

TABLE FOR POROSITIES OF VARIOUS ROCKS, VIBRATORY COMPACTION

Type of Rock

Cycles	CL-4	CL-.4	CL-.04	CG-4	CG-.4	CG-.04	RG-4	RG-.4	RG-.04	Marb.
↓	41.00	41.99	41.86	40.56	41.82	40.18	40.93	41.40	40.14	40.44
	39.90	40.53	41.31	41.10	40.87	41.53	39.00	40.44	40.34	40.67
1	39.54	39.77	40.56	40.56	41.63	41.00	39.36	40.64	40.73	40.67
	39.36	40.31	40.56	40.56	42.19	40.46	39.76	41.02	39.54	40.21
	40.27	40.53	41.13	39.56	40.48	41.27	40.00	40.64	40.00	40.21
	38.61	40.53	40.56	40.38	40.28	40.18	40.54	40.44	39.14	39.04
	39.54	39.58	40.37	39.80	39.89	39.63	38.51	39.85	38.72	39.04
10	38.42	38.99	39.60	38.71	40.87	40.46	38.71	39.65	39.34	40.21
	39.36	39.58	39.79	39.65	40.28	40.18	39.51	39.85	39.02	39.80
	38.22	39.38	39.99	38.71	39.68	40.18	38.30	39.85	38.72	39.51
	37.25	37.38	37.57	37.54	38.66	38.48	36.88	38.21	37.24	38.07
	36.45	37.79	37.57	36.94	38.03	38.19	37.32	38.42	37.02	38.56
100	36.45	37.05	36.94	36.94	39.08	38.48	36.21	38.21	37.24	38.07
	36.85	37.17	37.10	37.33	38.87	37.40	36.06	38.42	37.24	38.56
	37.45	37.38	36.94	36.54	37.82	38.48	37.10	38.42	37.08	38.31
	36.45	36.96	36.50	35.28	37.29	36.06	36.20	37.00	37.24	37.50
	35.42	35.23	35.85	35.71	36.06	37.90	35.97	36.26	35.91	37.32
1000	35.21	36.11	36.72	35.71	36.95	37.60	36.20	36.92	36.14	38.31
	36.45	36.54	34.95	35.50	37.39	35.74	35.97	36.70	36.58	36.81
	35.63	35.89	35.85	35.28	36.06	36.68	36.42	36.36	37.24	37.57

Discussion

W. H. CAMPEN, Omaha Testing Laboratories—This paper is intriguing, especially the method of determining the volume of the voids in rough surfaces. From the standpoint of one who has designed many mixtures over a period of almost fifty years, I wish to ask two questions.

What practical application can be made of the results obtained by the packing volume concept in the design of ordinary mixtures?

I agree that the angularity, surface roughness and asphalt absorption affect the asphalt requirement, but is it not a fact that in designing mixtures using residual air voids as a criterion, the affects on the asphalt demand are automatically taken into account if the asphalt absorption of the aggregate is taken into consideration?

EGONS TONS and W. H. GOETZ, Closure—Some practical applications of the results obtained so far are given in the paper just before the conclusions. The main purpose of this study was to attempt a unified procedure in "grading" different types of aggregates, including properties such as particle roughness, angularity and geometry in a quantitative manner. The work done so far is concerned with three one-size aggregates in the coarse aggregate range.

The findings show that rocks such as crushed limestone, crushed gravel, and rounded gravel, when graded according to a given particle packing volume, will compact to the same bulk volumes and identical packing porosities. This is usually not true with different aggregates from identical sieve sizes.

The packing volume concept also promises to give an analytical way of separating the two kinds of asphalt in a mix: (a) so-called stagnant asphalt which mainly fills voids in the particles (including surface roughness), and (b) the asphalt which binds the particles together and participates in flow when the mix is subjected to forces. Preliminary laboratory work indicates that the packing volume grading approach makes it possible to predict analytically the amount of asphalt needed for a rounded gravel aggregate and a crushed limestone to give the same "strength" or flow resistance under given conditions. Presently we have to use a definite design method, such as Marshall, and have to get our results by trial and error and without understanding what actually goes on.

The answer to the question is a qualified "yes". However, mix design in this sense consists of first choosing an aggregate and aggregate grading, guessing the asphalt contents, and then making specimens and testing to see where the optimum lies. If the combination of ingredients does not give the specified or desired properties (stability, voids, etc.) you have to choose again and start all over. What we are attempting here is to find out what parameters, including gradation, are important and how they can be measured so that we can establish an analytical way of designing a mix in the true sense. We also hope to be able to change and optimize mixes and to predict the behavior of compositions such as bituminous mastic concrete. Such mixes have little or no voids and residual air voids design criteria do not apply.