Skid Resistant Pavements in Virginia

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This paper reviews briefly the previously published works of Moyer, Shelburne, Sheppe, and others on the subject of skid resistance characteristics. It discusses several different methods of test, and describes the method most commonly used in Virginia, the measurement of stopping distance of a passenger car with wheels locked. A discussion of the factors affecting the accuracy of this method is included.

The results of stopping distance tests made at several hundred locations in Virginia are presented. These test results are tabulated in different ways to indicate, so far as possible, the effects of age, traffic, and, most particularly, the type of aggregate used in the mix. The data point very strongly to what is felt to be a rather serious lack of skid resistance on the part of most bituminous and even portland cement concrete pavements when constructed with limestone aggregates.

A description of the experimental program designed to determine economical ways of providing non-skid pavement surfaces is given. The purpose of the experimental program was (1) to find economical ways of deslicking existing roads, and (2) to find economical ways of building-in permanent high skid resistance at the time of construction. Skid test results on the eight experimental sections are presented and discussed. The conclusions reached are tentative pending additional service life.

● MEASUREMENTS of skid resistance on Virginia pavements began in 1946, shortly after the establishment of a research unit within the Department of Highways. The earliest work consisted of basic studies of the frictional resistance of various pavement surface types to forward skidding of an ordinary light passenger car. Tests were made at speeds of 10, 20, 30 and 40 mph on both wet and dry surfaces. Four tire conditions were investigated: synthetic rubber and natural rubber, each with good treads and worn treads.

Stopping distances were measured from the mark made by a chalk pellet, fired with a .22 caliber blank cartridge set off by an electrical connection to the brake pedal, to the final resting place of the car after completion of the skid. The average coefficient of friction between the tires and the pavement was computed from the formula:

$$f = \frac{V^2}{30D}$$

where

f = average coefficient of friction;
V = initial speed in mph at the instant of application of brakes;
D = stopping distance in feet.

Results of these early tests in Virginia were tabulated and presented in the form of a report to the Highway Research Board in December, 1947, by T. E. Shelburne and R. L. Sheppe (1). Among the principal conclusions reached in this 1947 report were these:

1. That stopping distances were of significance only on pavements in a wet condition.
2. That surfaces with sandpaper-like texture gave the shortest stopping distances, as opposed to surfaces glazed by an excess of bituminous material which gave the longest.

The 1947 report failed to indicate, except for one brief mention, that the polishing of certain aggregates might have any marked effect on slipperiness. In the 32 locations tested, the few relatively long stopping distances recorded seemed to be the result of excess asphalt rather than aggregate polishing, although there were six locations where the coarse aggregate consisted entirely of limestone.

A Major Cause of Slipperiness

Since the 1947 report, as will be brought out later, it has been indicated that the polishing action of traffic on pavements constructed with limestone aggregates is a major cause of poor skid resistance, even when there is no excess of asphalt and even when the mix has a fairly coarse and open texture. More recent tests have found quite definitely that, except for those surfaces glazed by excess asphalt, the great majority of slippery pavements in Virginia were constructed of limestone. This situation has become one of great concern to highway engineers in Virginia, in view of the fact that in three of the eight construction districts in the state, practically all of the aggregates quarried are of a limestone or dolomite character.

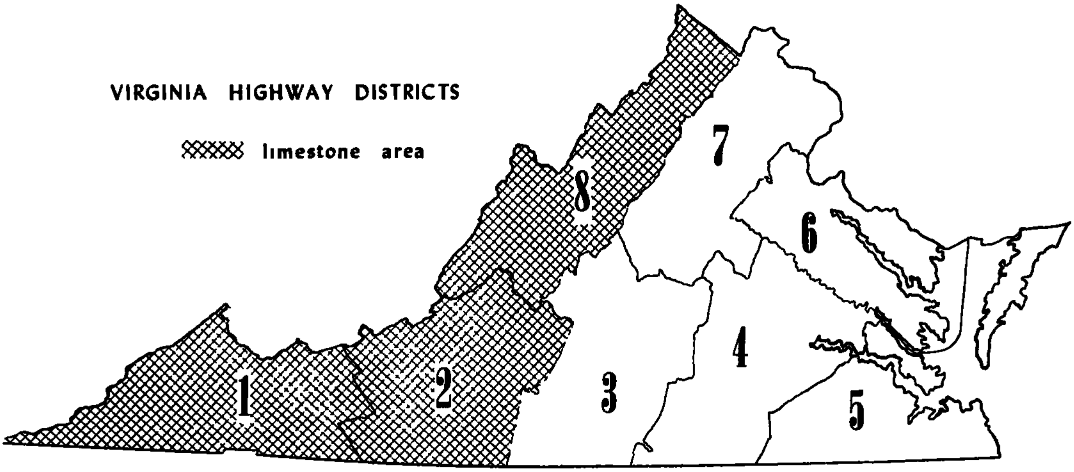

Figure 1.

To emphasize the fact that pavements built with limestone aggregates are more slippery than any others in Virginia, Table 1 has been prepared to show the geographical distribution of accidents involving skidding on wet pavements. The westernmost districts (1, 2, and 8 in Figure 1) are the three districts in which nearly all the road construction aggregates are limestone or dolomite. Some limestone is shipped from District 2 into parts of District 3, but in Districts 4, 5, 6 and 7, no limestone is quarried, and practically none is ever used.

A glance at the figures in Table 1 will show that the frequency of skidding accidents on wet pavements in the three limestone districts, on the average, is around twice the frequency in the other five districts. Similarly, the amount of property damage resulting from skidding accidents in these districts is nearly $2\frac{1}{2}$ times that in the

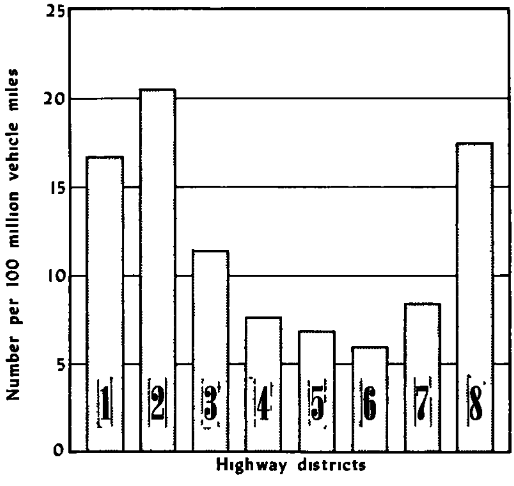

Figure 2. Skidding accidents.

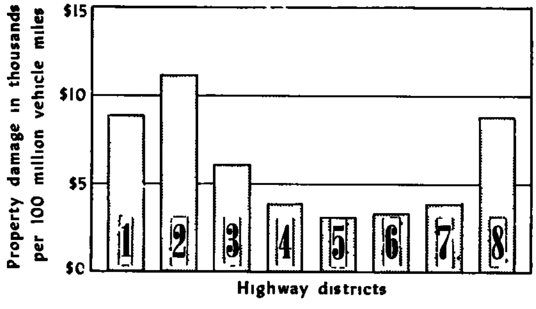

Figure 3. Skidding accidents.

TABLE 1[a]

VIRGINIA ACCIDENT DATA, 1953-1954

Highway Districts	Skidding Accidents per 100 Million Vehicle Miles	Property Damage from Skidding Accidents per 100 Million Vehicle Mi	All Accidents per 100 Million Vehicle Miles
1. Bristol	16.7	$ 8,839	307
2. Salem	20.5	11,019	331
3. Lynchburg	11.4	6,006	312
4. Richmond	7.6	3,922	256
5. Suffolk	6.8	3,216	387
6. Fredericksburg	6.0	3,418	304
7. Culpeper	8.4	3,907	350
8. Staunton	17.5	8,848	279
State-wide	11.2	$5,725	320

[a] Data computed from figures furnished by the Division of Traffic and Planning of the Virginia Department of Highways from IBM punch cards prepared for each accident. Figures related only to the rural primary system.

other five, when related to total vehicle miles traveled.

In distinct contrast, the frequency of accidents of all types follows an entirely different pattern; the three limestone districts which run first, second, and third in frequency of skidding accidents are seen to run third, seventh, and fifth respectively in frequency of all accidents.

Other theories to explain the preponderance of skidding accidents in the westernmost districts might be advanced. These districts are admittedly more mountainous, and many skids could result from failure to negotiate mountain curves; however, the principal artery traversing all three districts, U.S. Route 11, follows the valleys for almost its entire length. Proponents of portland cement concrete pavements may claim that the almost complete absence of concrete roads in these districts is the explanation, but a glance at figures to be presented later will show that even concrete can become quite slippery when constructed with limestone aggregates. It seems unlikely that any combination of other factors could be the cause of so much higher accident frequencies in the Bristol, Salem and Staunton Districts.

As a result of the concern of the Highway Department over the slippery pavement condition in the limestone areas, the Virginia Council of Highway Investigation and Research as early as 1950 began a program of skid testing to determine the relative benefits of various methods of deslicking pavements which were otherwise in satisfactory condition. Since that time considerable effort has been exerted towards both economical deslicking of existing pavements and building permanent skid resistance into new pavements at the time of construction.

A full discussion of the effectiveness of the various methods which have been used to skid-proof Virginia's pavements is part of the dual purpose of this report, and will be found in Part II. The other purpose is a description of the test methods currently in use, with some discussion of their reliability, and a summary of the results of the rather broad program of testing.

Part I
Recent Skid Testing in Virginia

Skid resistance measurements reported here have been gathered over a period of about 18 months for three principal purposes.

First, as part of their routine investigation of accidents, the Traffic and Planning Division often requests that skid tests be performed at the scene of serious skidding accidents. As a result of these tests, particularly where they indicate that the road

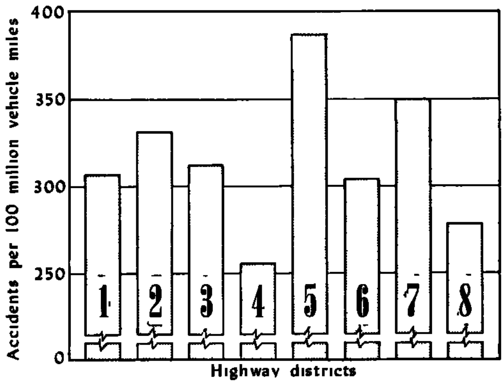

Figure 4. All accidents.

may be relatively slippery, a recommendation is often made to the Maintenance Division that a deslicking treatment be applied.

Secondly, in connection with proposed field experiments in which certain non-polishing aggregates were to be added to limestone mixes to prevent their becoming slippery, the Research Council in 1954 began a series of exploratory skid tests aimed at determining polishing characteristics of aggregate from various sources. Previous records were found to include those of tests on pavements constructed from relatively few of the sixty or seventy major sources of aggregate in Virginia. Consequently, an effort was made to secure some idea of the skid resistance properties of pavements constructed from every major type of aggregate used, and, in the case of the limestone type, from as many individual sources as possible.

A third series of skid resistance measurements, from which data for this report were drawn, was made in the fall of 1954, on authorization from Burton Marye, Jr., then Chief Engineer of the Virginia Department of Highways, who ordered the performance of tests on every mile of the heavily traveled Class I and II roads in the primary system which, in the opinion of the resident engineers, might have questionable skid resistance. This was one phase of a comprehensive accident prevention program dealing also with such features as superelevation of curves, condition of shoulders, adequacy of signs and pavement marking, and obstructions to sight distance. In connection with this program, skid tests were made at hundreds of locations on all types of surfaces scattered over all eight of Virginia's highway districts.

TEST METHODS

A number of different methods of measuring the resistance of pavements to skidding, both forward and sidewise, have been reported by other states and many foreign countries. Experience in Virginia has been confined to one rather widely used method and, quite recently, to a very simple method not so widely used.

Stopping Distance Method

All tests reported here were conducted by the same stopping distance method reported by Shelburne and Sheppe in 1947, except that the test has since been streamlined for speed. Pavements are no longer tested in a dry condition; also, testing at very low speeds has been discontinued. Only good tread tires are used.

The time required to complete the testing at a given site has been greatly reduced by the adoption of the Wagner Stopmeter and fifth wheel, by means of which the initial speed and stopping distance may be measured from dials within the test car. These instruments are connected to the brake light switch so that at the instant the brakes are applied, the initial speed is locked on the speedometer and the odometer begins measuring the length of the skid. With this system, after warning signs have been placed and flagmen stationed, two runs at 30 mph and three or four at 40 may be made in about fifteen minutes or less.

Standard Test Speed 40 mph. Tests at most sites have been made at 40 mph only. However, a good many tests have been made at 30 mph, for one of two reasons. First, when there is any question of the safety of running the test at 40 mph, one or two runs are made at lower speeds to get the feel of the test site. Secondly, at 66 of the most recent locations tested runs were made at both 30 and 40 mph to indicate the difference in the coefficients of friction from the two speeds.

Here it was found that, on the average, the coefficients of friction computed by the standard formula:

$$f = \frac{V^2}{30D}$$

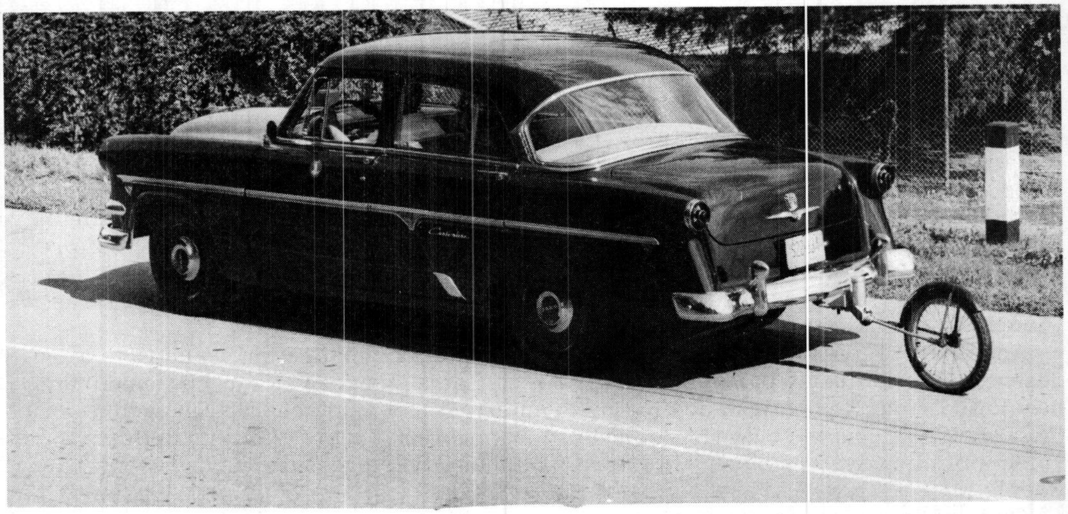

Figure 5.

for skids from 30 mph were 1.07 times higher than those from 40 mph. There was a rather wide range in the values of this relationship, however, since the coefficient at 30 mph varies from 0.93 to 1.17 times that at 40.

This being the case, it has been our feeling that results of tests run at 30 mph or less were not truly indicative of those which might be obtained from tests at higher speeds. Consequently, where safety permits, all tests are made at as near to 40 mph as possible, and results are reported in terms of stopping distances from exactly 40. In cases where the actual test speed varied by one or two mph from 40, an adjustment is made by computing the coefficient of friction and substituting this coefficient back into the formula with a value of 40 for V to secure a corrected stopping distance.

Factors Influencing Accuracy and Reproducibility of Results. This paper does not report results from a program designed specifically as a research study. Admittedly there are factors influencing the results which were not controlled. Four different sets of tires were used, and results were accumulated from tests at all seasons of the year. The effects of these factors, however, are felt to be diminished to a great extent by the volume of data amassed. Instead of only 32 test sites, as reported in 1947, this paper reports results from 262 different locations.

Examination of the data might indicate that tires "A" which were furnished on the 1954 Ford test car when it was new had the poorest skid resistance of all, while tires "D," used in the latest tests, had the best. This may well be due, however, to the generally low temperatures at the time tires "D" were in use, contrasted with summer temperatures for tires "A," rather than to any great differences in tread design or composition of the rubber. No conclusions can be drawn here on the full effect of the tire or temperature variable.

Films of oil and dust are generally recognized to have a marked effect on the stopping distance although authorities are not in complete accord on this (2). No attempt will be made in this report to evaluate this effect.

Wherever possible, skid tests should be made at locations where the percent grade is practically zero. Occasionally, such a location cannot be found, and in such cases some correction must be made. One method is the use of a modified form of the formula for computing the coefficient of friction, as follows:

$$f \pm \frac{\%\ \text{grade}}{100} = \frac{V^2}{30D}$$

An alternate method is the performance of tests both downgrade and upgrade and averaging the results. This is the method most usually used in Virginia, though the occasion to do so seldom arises.

Non-uniform surfaces may have the greatest effect on reproducibility. Bituminous

surface treatments which have developed fat spots, particularly in the wheel tracks, are most difficult to test by the stopping distance method. Often these fat spots may be dangerously slick, but are of insufficient size for accurate skid resistance measurement.

In connection with skid resistance of surfaces which are glazed by excess bituminous material, it may be of interest to note here a rather definite impression that such surfaces are apt to be more slippery in cold weather when this type of surface is glassy hard than in hot summer weather when it may be soft and gummy enough to create more friction. This is in contrast to observations on other surfaces and to findings of previous investigators (2, 3), all of which indicate an increase in skid resistance in the winter months, which is possibly the result of a decrease in the density of oil films in winter.

Any delay in locking the wheels of the car after brake application would have a tendency to shorten the stopping distance. It is a generally accepted fact that maximum deceleration can be obtained by braking to an extent just short of that required to lock the wheels (3), and the automotive industry has published reports of their efforts to develop a braking system which will exert maximum brake pressure without locking the wheels (4).

Quite by accident, an outstanding example of this fact was developed in a recent test with an inexperienced driver at the wheel. His first test run resulted in a stopping distance of 99 feet from 40 mph, but an observer noted that the wheels never did lock during the entire run. On the five succeeding runs at the exact same location, the driver was careful to lock the wheels each time, and stopping distances ranged from 116 feet to 126 feet.

The test car used in Virginia has no special device to insure instantaneous locking of all four wheels. It has been our observation that when the brakes are well adjusted and the driver exercises due care, brakes may be locked in a very small fraction of a second, and variations in this time lag would have no significant effect on the measurements.

Decelerometer Method. Some months ago the Research Council acquired a Tapley

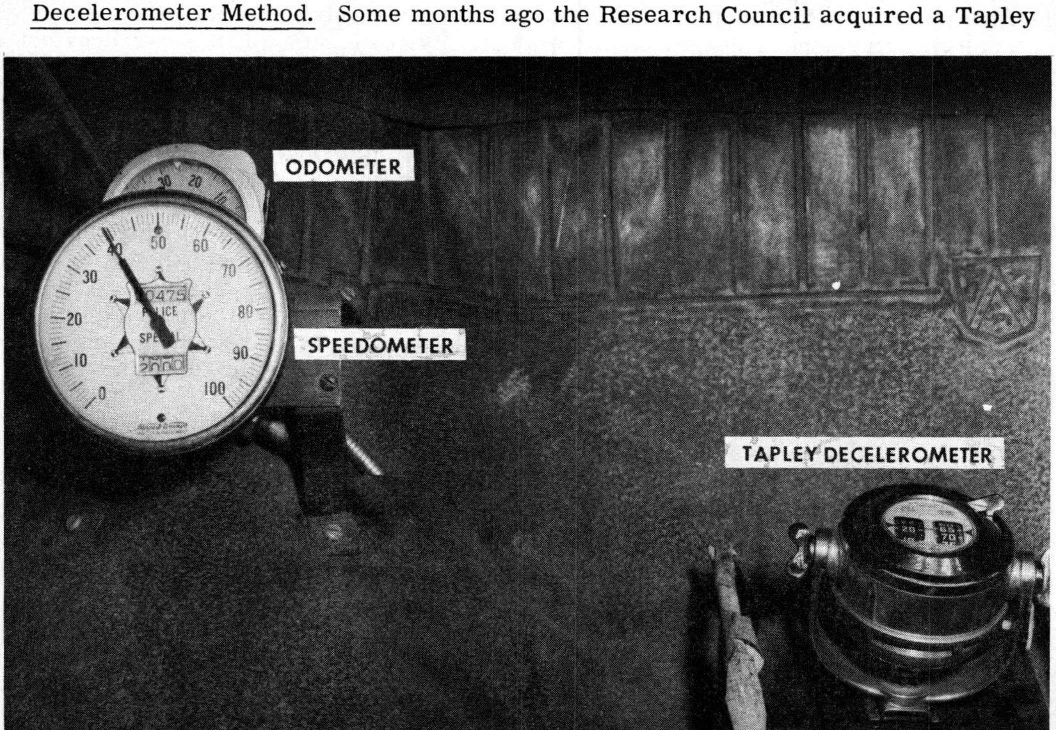

Figure 6.

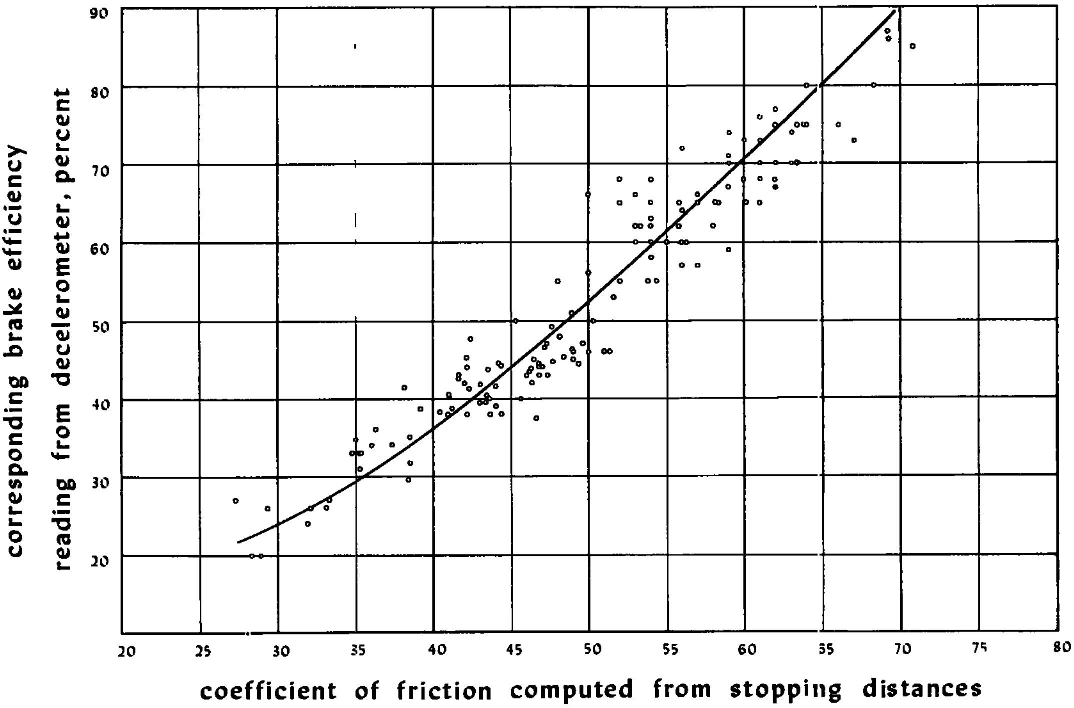

Figure 7. Coefficients of friction versus decelerometer readings on same test run.

Decelerometer, an instrument designed primarily for testing brakes (see Figure 6). This instrument works on the principle of a damped pendulum which swings forward from its normally level position in an arc the angle of which is proportional to the rate of deceleration. The units of measurement indicated by the meter are, on the left hand dial, feet to stop from 20 mph, and on the right, brake efficiency in percent. This latter figure has been found to correspond exactly to the coefficient of friction computed from the stopping distance shown on the left dial, multiplied by 100.

To explore the usefulness of this very simple device, readings of the right dial, showing brake efficiency, were taken at a number of locations simultaneously with measurements of stopping destance. This method of reading the decelerometer introduced difficulties, some of which might be avoided by adhering closely to the manufacturer's instructions. When the decelerometer dials are set in the "test" position, the readings remain visible after the test is completed. The manufacturer recommends that the car not be brought to a full stop, but that the brakes be applied only long enough to secure the reading and then released. To save time, however, on our testing program the decelerometer readings were made by an observer seated beside the driver during regular stopping distance runs. The difficulty was that as the sliding car slowed down below about 25 mph, the deceleration rate increased and the dial readings crept higher and higher in an erratic manner until in some cases the final lurch caused the dial reading to register 100 percent.

It was found, however, that when the test was started from 40 mph, the brake efficiency reading usually remained fairly constant until the car had slowed down to about 25 mph before it began to creep up. This was the reading that the observer attempted to record. The results, along with the corresponding coefficients of friction, are shown in Figure 7.

At a few locations, the brakes were released in accordance with the manufacturer's recommendation, and the readings held on the decelerometer were found to be reproducible within about 3 percent and also to be in good agreement with previous readings caught during runs which carried to a complete stop.

TABLE 2

SUMMARY OF SKID RESISTANCE DATA BY MAJOR AGGREGATE TYPES

(All Tests from 40 mph on Wet Pavement)

I Type of Aggregate	II Type Tires	III No. of Sites Tested	IV Avg. Distance to Stop (feet)		V Range of Distances to Stop (feet)
Limestone	A	57	137		
	B	52	135		
	C	21	121		
	D	48	126		
	All	178		132	90 - 178
Granites	A	8	118		
	B	7	90		
	C	7	106		
	D	1	108		
	All	23		106	79 - 137
Trap Rocks (Diabase)	A	7	131		
	B	0	---		
	C	0	---		
	D	6	116		
	All	13		124	108 - 155
Gravels or Gravel & Granite Mixtures	A	0	---		
	B	5	97		
	C	0	---		
	D	0	---		
	All	5		97	87 - 103
Sand or Mixtures of Sand & Stone Screenings	A	4	114		
	B	9	92		
	C	2	95		
	D	2	95		
	All	17		98	77 - 125

From Figure 7 it may be readily seen that the brake efficiency does not follow the coefficient of friction from 40 mph, the higher readings on the meter falling above the coefficient of friction and the lower readings falling below. There is quite a scatter in the readings, but it should be borne in mind that both brake efficiency and computed coefficients of friction are inexact measurements, and possibly the decelerometer readings may be found to have better reproducibility than stopping distances.

It would seem that such a device could become a very useful tool for securing a fair idea of the comparative skid resistance of a large number of pavements in a short time. On a rainy day, measurements could be made with great rapidity and with no additional equipment. Also, it may be possible to secure readings on slippery spots too small to test by the stopping distance method.

RESULTS OF SKID RESISTANCE TESTS

As has been emphasized earlier, the data presented here were accumulated over a period of about 18 months. Four sets of tires were used during this period, and tests were made with pavement temperatures varying from barely above freezing to over 100 degrees.

While a great many results have been recorded from tests made on surface treatments, none of these are included in this report. The data presented here are intended to indicate principally the effect of certain aggregates, particularly limestones, on pavement slipperiness. Surface treatments too often have fat spots and other non-uniform conditions which might affect the stopping distance to a greater extent than the type of aggregate.

With this in mind, Table 2 has been prepared to include tests from 236 locations where the surface was a hot-mixed bituminous concrete of one type or another. This table is broken down by major aggregate types such as limestone (which includes dolomite), granite, trap rocks, gravels, and sands. The last category is made up entirely of F-1 sand asphalt mix, the most common type found in the eastern half of the state. The other categories consist entirely of the coarse aggregate mixtures, either the I-3, the H-2, or the old H-3, which was the forerunner of the present day I-3. These mixes were laid from 1942 to 1955 under specifications in effect at the time. A summary of these specifications is given as Appendix Table B.

TABLE 3

TESTS ON PORTLAND CEMENT CONCRETE PAVEMENTS
(Wet Pavements, 40 MPH)

Aggregate Type	No of Sites Tested	Average Stopping Distance (feet)	Range (feet)
Limestone	17	141	120-184
Natural Sand & Gravel	3	109	94-119

The periods when each of the four sets of tires were used for skid testing are shown below:

Tires A - June, July, 1954
Tires B - August - November, 1954
Tires C - August, September, 1955
Tires D - October - December, 1955

Thus it may be seen that Tires A and C were used in hot weather almost entirely, Tires B in hot to moderately cool weather, but Tires D only in cool to quite cold weather.

This, admittedly, is unfortunate. A number of the later tests with Tires D were made at or near the same sites as earlier tests in an attempt to determine how much the aggregate polishing had progressed in the last 15 to 18 months. It was found, however, that in most cases, the stopping distances had decreased rather than increased. The reason for this may be attributed to the change in tires or to the marked difference in temperature, or both, but at this time there is no way of knowing which had the greater effect.

Future tests for the next few years, particularly those on the special field test sections to be described in Part II, will all be run with Tires D. A set of four spare wheels and tires have been obtained for the test car so that the test tires can be removed and stored while the car is not being used for testing. In this manner the tire variable should not enter the picture in evaluating the test sections.

In spite of the tire variable, it should be fairly obvious from examining Table 2 that stopping distances on limestone pavements with any or all tires are significantly longer than on any other pavements. This is true even though at nearly half of the 178 sites shown in the limestone category, the mix was laid under a specification requiring at least 50 percent of the fine aggregate (or about 20 to 25 percent of the total aggregate) to be silica sand.

The results in Table 2 are further broken down in Table A of the appendix. This table considers each individual source of aggregate separately, and gives also, where it is known, the name of the geological formations encountered in the various quarries. A general description of the geological formations is also found in the appendix. This may be of use to readers from states other than Virginia who may wish to investigate the skid resistance of similar aggregates in their states.

Lest it be assumed that the poor skid resistance of limestone aggregate plant mixes is due to causes other than the aggregate, the information in Table 3 is submitted to show the same effect when limestone aggregates are used in portland cement concrete pavements.

The 17 locations of the limestone category were all in two adjacent projects on Route 11 north of Roanoke. Two different sources of coarse aggregate were used and a number of different sources of fine aggregate. In general, around 50 percent of the fine aggregate was limestone sand and the balance natural silica sand. These projects were about 15 years old when tested. The three locations in the natural sand and gravel category were in three entirely separate areas, and ranged in age from about five to twelve years.

Realizing then that pavements constructed of limestone do result in significantly

longer stopping distances than those of any other aggregate common to Virginia, the question naturally comes up: "What should be considered a reasonably safe stopping distance?"

CRITERIA OF ACCEPTABLE MAXIMUM STOPPING DISTANCE

For several years, the figure of 113 feet from 40 mph on wet pavement, exclusive of reaction time, was considered in Virginia to be the standard safe stopping distance. This figure was found in the AASHO's suggested design standards for minimum stopping sight distances, published in 1940 (5). More recent design standards (6) are based on a coefficient of friction of 0.33, which corresponds to a stopping distance of 161 feet from 40 mph. However, it should be borne in mind that these figures are intended to encompass nearly all significant surface types and field conditions, including the combination of worn tires and polished (but not bleeding) surfaces. The AASHO does not set up acceptable or even desirable stopping distances; instead, it recognized that slippery pavements do exist and attempts to allow for them in establishing minimum sight distances.

Realizing fully that numerous variables do affect the results of the tests, it would be foolish to draw a fine line above which stopping distances should be called unsafe and below which safe. Obviously, if this were done, then each time the tires were changed on the test car, pavements which might have been called unsafe would suddenly become safe, or vice versa.

Instead, the boundary between safe stopping distances and those which would be recognized as definitely hazardous should be a rather broad band. Its limits would of necessity be governed to a considerable extent by the economics involved in attempting to correct all the pavements which would fall into the unsafe category.

Virginia's 1955 "deslicking" program, to be described in Part II, was based on an assumed safe stopping distance of 133 feet from 40 mph, corresponding to a coefficient of friction of 0.40. This was an arbitrary figure, but one which has been suggested for use by several previous investigators. Pavements which tested very close to this figure were regarded with suspicion; those which tested as much as 10 feet above it were in practically all cases given the deslicking treatment.

The various experimental methods of increasing skid resistance of existing pavements and building permanent skid resistance into new pavements, which will be described in the next section of this paper, will not be considered successful unless they are found to produce stopping distances substantially below the 133-foot figure, certainly below 120 feet.

Part II
Experimental Investigation of Measures for Increasing Skid Resistance

It has been shown that pavements containing limestone as aggregate often become polished. The Virginia Department of Highways realizing this has carried on deslicking programs for several years. The most extensive was in 1955 when approximately $450,000 was spent to deslick roads. Although Virginia's anti-skid programs have been costly, especially when it is realized that the deslicking treatments add nothing to the structural strength of the road and little to the durability of the pavement, the Department has felt obligated to pursue them. As a result it was decided that some research should be undertaken to determine the most economical method of deslicking and further to determine what steps would be necessary to "build-in" permanent skid resistance at the time the pavements are constructed. In accordance with this an experimental program has been undertaken.

PURPOSES OF EXPERIMENTS

The specific objectives of the experimental program were twofold. The first objective was to find the most economical method of building-in a permanent skid resistance at the time of construction. The second was to develop an economical method of de-

slicking existing pavements whose only deficiency was the lack of adequate skid resistance.

Basically the problem reduces to one of economics since it is well known that high skid resistant bituminous roads can be constructed with certain materials. The sand asphalt type of pavement, used in Virginia in the Coastal Plain section, is excellent in this respect. Mixes made from such polish-resistant aggregates as crushed gravels and granites also yield high skid resistant surfaces (see Table 2). In the interest of economy, however, it is essential that limestone constitute the bulk of the aggregate that goes into road making in the three westernmost districts of Virginia. The essence of the problem, then, is the determination of how small an amount of non-limestone aggregates is sufficient to provide skid-resistant surfaces in these areas.

To investigate this problem a total of ten series of test sections was placed. These are summarized in Table 4.

BUILDING-IN HIGH SKID RESISTANCE

The most frequently used asphalt concrete pavement in Virginia is designated in the specifications as Type I-3. In attempting to improve the skid resistance of pavements containing limestone aggregates, no attempt has been made to alter the I-3 grading specification. Rather, the experiments were directed toward adding abrasive materials to this type of mix with little or no changes in the gradings. Although it is known that surface texture is an important aspect of skid resistance, some experimental work in Virginia (to be described later) has suggested that 100 percent limestone surfaces will polish regardless of the size of the stone used. Therefore, no experiments have been devoted to improving the skid resistance of 100 percent limestone mixes by altering the surface texture. It is possible, however, that the addition of polish-resistant aggregate in a given quantity and type may result in differential improvements depending upon the resulting surface texture. The present series of tests has not studied this facet of the problem.

Adding Polish-Resistant Fine Aggregate

In 1954 two series of test sections were placed in which the amount of silica sand was varied in the I-3 grading. The skid test results are shown in Table 5. Also a summary of skid test measurements on mixes in which 20-25 percent sand was added as a part of the regular paving program in 1953 is shown in Table 6.

It can be noted that at the time of testing the experimental pavements in Table 5 had been under traffic for only a year and, furthermore, the traffic count is not high. The data show that the addition of sand to the mix on Route 33 has improved the skidding resistance, although the benefit is relatively slight. The low traffic count on Route 21 is probably the reason for the high skid resistance on all of these sections. A better esti-

TABLE 4

1954-1955 EXPERIMENTAL PROGRAM - PROVIDING SKID RESISTANT ROADS

BUILDING-IN AT TIME OF CONSTRUCTION		DESLICKING AFTER PAVEMENT BECOMES SLICK	
Adding Polish-Resistant Coarse Aggregate (Three Sections Placed - 1955)	Adding Polish-Resistant Fine Aggregate (Two Sections Placed - 1954)	Applying Thin Surface of Silica Sand Plant Mix (One Section Placed - 1955)	Fine Sand Plant Mix (Four Sands Tested - 1955)
1 GRANITE-LIMESTONE, Rte 11 Montgomery County	1 SILICA SAND-LIMESTONE Rte 11 Rockingham County	1 SILICA SAND, Rte 11 Shenandoah County	1 SAND A, Rtes 11 & 250 Augusta County
2 GRAVEL-LIMESTONE, Rte 11 Botetourt County	2 SILICA SAND-LIMESTONE, Rte 21 Wythe County		2. SAND B, Rte 11 Botetourt County
3 GRAVEL-LIMESTONE, Rte 11 Rockingham County			3. SAND C, Rte 11 Botetourt County
			4. SAND D, Rte 11 Botetourt County.
(The Polish-Resistant coarse aggregate was added in the amount of 0, 10, 20 30% of total aggregate)	(The silica sand was added in the amounts of 0 25, and 50% of the total aggregate)	(Applications of from 1/4 - 1/2 inches were placed)	(Asphalt varied between 6-8% Hydrated lime and powdered rubber added to some, nothing added to others)

TABLE 5

SKID TEST RESULTS ON MIXES CONTAINING
POLISH-RESISTANT FINE AGGREGATE

RTE	County	Traffic Count VPD	Sand % Total Aggregate	STOPPING DISTANCE, 40MPH 1 mo. (feet)	9 mos. (feet)	13 mos. (feet)
21	Wythe	1102	0	103	102	110
			20-25	91	93	108
			40-50	91	99	109
33	Rockingham	3549	0	---	123	134
			20-25	---	109	115
			40-50	---	110	122

TABLE 6

SKID TEST RESULTS ON MIXES CONTAINING
20-25% SILICA SAND

	With 20-25% Sand					Without Sand				
Mix Type	No. of Sites	No. Cases Over 133'	% Cases Over 133'	No. Cases Over 120'	% Cases Over 120'	No. of Sites	No. Cases Over 133'	% Cases Over 133'	No. Cases Over 120'	% Cases Over 120'
I-3	60	21	35	35	58	23	14	61	21	91
	1-3 yrs. old at time of test					1-8 yrs. old at time of test				

mate of the effectiveness of the sand on both of these routes will be possible after they have been used for several years.

The results reported in Table 6 were obtained from pavements placed in the Department's regular resurfacing program rather than as a part of an experimental project. All I-3 mixes placed in 1953 and 1954 that utilized limestone aggregates were required to include 20-25 percent silica sand. Sixty sites have been tested and, as shown in Table 6, 21 of these (35 percent) had stopping distances over 133 feet and 35 (58 percent) over 120 feet. Although the pavements with sand as shown in Table 6 are not strictly comparable to those without sand, because of the difference in the ages, they do provide an insight into the benefit to be gained from adding sand. Thus it appears that sand does improve the skid resistance of some limestone pavements, but since a high percentage are above 120 feet the addition of 20-25 percent silica sand is considered an inadequate answer to the problem of building-in permanent high skid resistance in the I-3 type surface.

Adding Polish-Resistant Coarse Aggregate

Subsequent to the extensive testing of the limestone pavements containing 20-25 percent silica sand, it was decided that a different approach to building-in skid resistance was necessary. It was assumed that possibly the ineffectiveness was due to the position of the sand within the mix; the tires were riding on the coarse limestone particles

rather than on the sand. It was thought, therefore, that the addition of a polish-resistant coarse aggregate[1] might provide a better solution. To determine whether or not this was true, three test sections were placed in 1955.

The skid results obtained shortly after the test pavements were placed are shown in Table 7. The results are not meaningful because the pavements were too new. A better estimate of the effectiveness of the polish-resistant coarse aggregate can be made at ages of one and two years.

TABLE 7

SKID TEST RESULTS ON MIXES CONTAINING POLISH-RESISTANT COARSE AGGREGATE

Rte	County	Traffic Count VPD	Polish Resistant Coarse Aggregate Type	Polish Resistant Coarse Aggregate % of Total Agg.	STOPPING DISTANCE (Feet)	Age Mos.
11	Montgomery	8201	Granite	0	128	2
				10	115	
				20	103	
				30	107	
11	Botetourt	7438	Crushed Gravel	0	91	1
				15	97	
				20	98	
				25	95	
11	Rockingham	4886	Crushed Gravel	0	138	2
				10	130	
				20	128	
				30	129	

TABLE 8

SKID TEST RESULTS ON DESLICKING TEST SECTIONS

Sand	Asphalt Content %	Grade of Asphalt (pen.)	Hydrated Lime Filler %	Rate of Application, psy	STOPPING DISTANCE, 40MPH	Age
A	6 to 7	85-100	2-1/2 to 3-3/4	10-15	85'	1 - 3 mos.
B	6-1/2	85-100	2-1/2 to 3-3/4	10-15	91'	
C	3	138	1-1/4	10-15	89'	
D	7-3/4	138	1-1/4	10-15	96'	
Rock Asphalt	-	-	-	10-15	90'	

[1] The term polish-resistant coarse aggregates as used here includes granites and high quartz gravels.

Thin Plant Mix Sand Application

Another alternative proposed for building-in skid resistance was the application of a thin plant mix surface made from silica sand. Such a mix could be applied at the time of construction over the regular limestone aggregate layer. Since silica sand is fairly expensive in the limestone areas, it was desirable to utilize as little as possible. A test section was placed primarily to determine how thin an application could be applied and what its durability would be. The mix made from a fine sand, 2½ percent hydrated lime, and 6 percent asphalt, was fabricated at an asphalt plant and applied with a Barber-Greene paver on a section of Route 11 in Shenandoah County. The sand used (Sand A) was also utilized in deslicking experiments and is described in Table 8.

It was found that the mix could be applied satisfactorily in layers as thin as ¼ inch. At an age of four months the ¼-in. layer is performing satisfactorily, but a determination of the durability of the mix will not be possible for several more years. On this section a stopping distance of 77 ft at 40 mph was obtained. Experience has shown that very few pavements exhibit skid distances less than 80 ft, so the skid results are excellent.

DESLICKING MIXES

The polishing of limestone aggregates has necessitated the application of deslicking treatments. The need for information on the effectiveness of various methods of deslicking was first realized in 1950, at which time several experimental sections were placed. The test sections consisted of (1) rock asphalt, (2) precoated silica sand (3 percent asphalt), (3) precoated limestone sand (3 percent asphalt), (4) limestone seal treatment, (5) slag seal treatment.

All test sections embodying limestone in any form became slick within a year or so. The precoated silica sand mixes yielded a high skid resistance but soon wore off the pavement. The slag seal and the rock asphalt were found to be the most successful. Both sections exhibit stopping distances less than 110 ft after five years of use. Subsequent to this study the Department has used rock asphalt extensively for deslicking purposes.

Experimental Mixes of 1955

Test results have shown that rock asphalt (sandstone type) is very effective in reducing slickness. In the interest of economy, however, a series of tests was planned to lead to additional suitable methods of deslicking. The emphasis was on the use of local sands which, it was hoped, would result in lower deslicking costs.

The fine sand deslicking treatments placed were hot plant mixes and used penetration grades of asphalt. The four sands used in the experiments were applied in the same manner as rock asphalt. They differed from the 1950 precoated sand mixes in that they embodied finer sand, a higher asphalt content, and hydrated lime.

The mixes and their components are described in Tables 8 and 9. The mixes described in Table 8 do not constitute all the combinations of asphalt, filler, and sand that were placed on the road but only those that appear to be the best design for each sand. Photomicrographs of the sand grains in rock asphalt and three of the test sections are shown in Figures 8 and 9. For comparative purposes photomicrographs of Ottawa sand and a commercial Coastal Plains sand are included in Figures 10 and 11, respectively. There appears to be only slight differences in the particle shape of the sands used in the experimental deslicking mixes and the rock asphalt grains. The Ottawa sand is much more rounded than any of the others.

TABLE 9

GRADINGS OF SANDS USED IN EXPERIMENT

Sieve No	Sand A % Passing a	Sand B % Passing b	Sand C % Passing c	Sand D % Passing d
4				100
10	100	100	100	98
40	93	85	62	45
80	14	25	15	6
200	1	2	2	3

[a] Crushed sandstone, washed, screened, sold to glass manufacturers.
[b] Crushed sandstone, washed, screened, impure batches of glass sand
[c] Crushed sandstone, washed, sold for mortar sand
[d] Unwashed sand bank

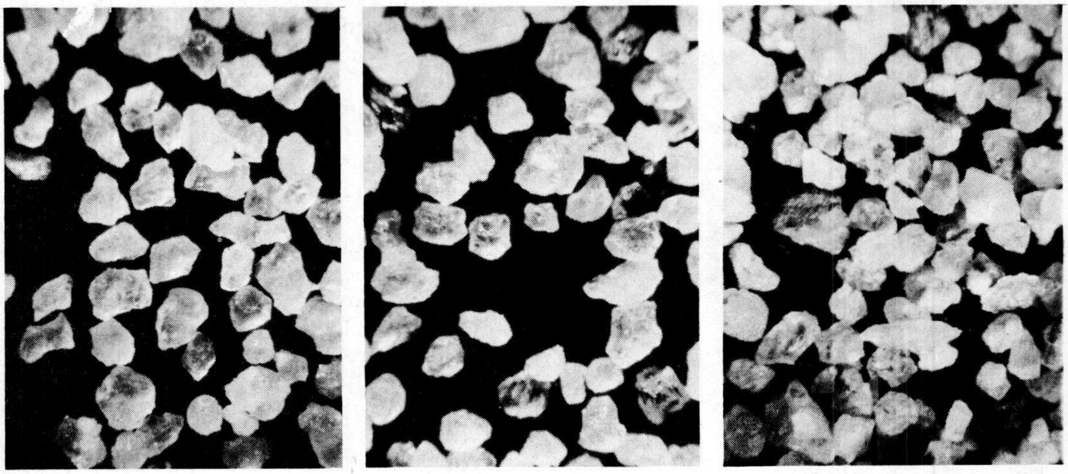

Figure 8. Sand grains in three test sections.

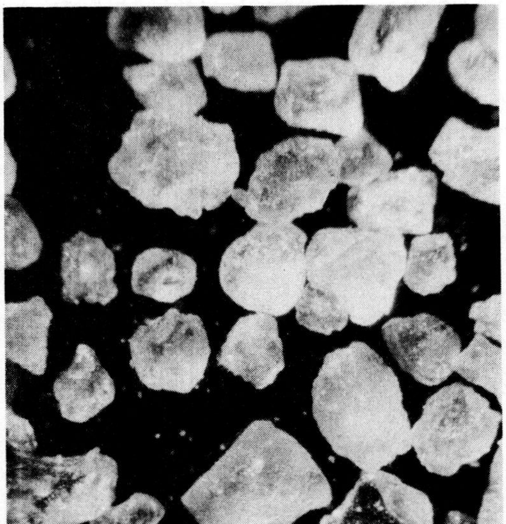

Figure 9. Sand grains in rock asphalt.

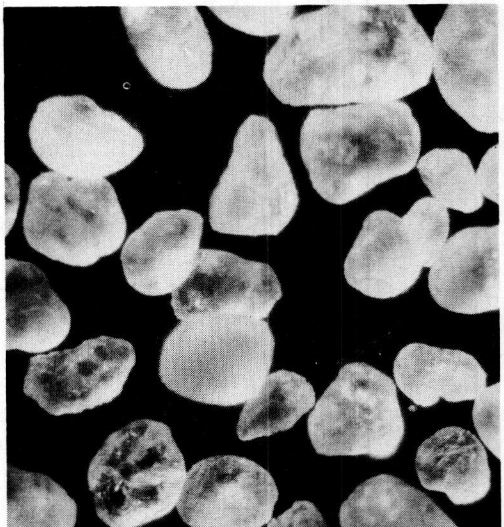

Figure 10. Ottawa sand.

Methods of Application

The Virginia Department of Highways for several years has used an Anderson spreader for placing rock asphalt. The equipment is shown in Figures 12 and 13. The same trucks are used to spread calcium chloride in the winter and were utilized for deslicking purposes because they were available.

The fine sand mixes were fabricated in the usual way at the asphalt plant. Generally a 1½- to 2-minute mixing time was necessary for a two-ton batch. The temperature of the mix was generally about 260-300 deg F.

When the deslicking material is being applied, it is fed to a spinning disc (Figure 12) by a screw feed located in the bottom of the hopper. This same equipment was used to

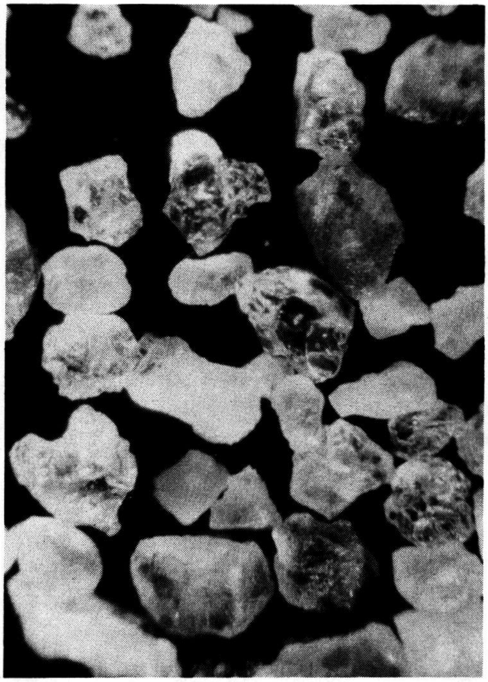

Figure 11. A commercial Coastal Plains sand.

install both the rock asphalt and the plant-made deslicking mixes. The rate of application was variable but was generally in the range of 10-15 psy. The spreaders were capable of applying the material at a rate of only about 5-10 psy per pass, so several passes were necessary. To provide a good bond it was found necessary to use a tack coat material that yielded a 100 percent coating over the area to be deslicked. An RC-0 was used at the rate of 0.1 gallons per square yard to accomplish this. It has been found that the tack coat is essential in holding the deslicking material in place, since the deslicking material itself has practically no ability to bond itself to the old surface.

The production rate of the rock asphalt is greatly influenced by time necessary to heat the material to application temperatures. In Virginia the material is placed in the truck hopper and heated by a system of steam coils mounted in the hopper. Utilizing four trucks it was found that about 60-80 tons could be applied per day, an amount that covers about ⅔ of a mile on a two lane

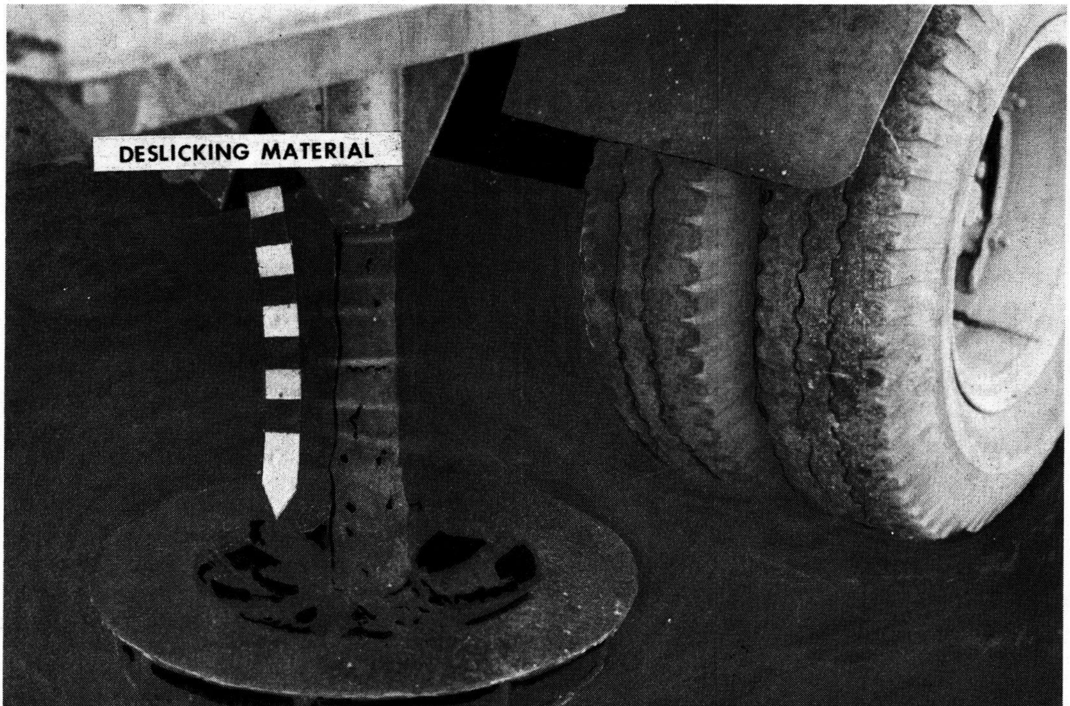

Figure 12.

Figure 13.

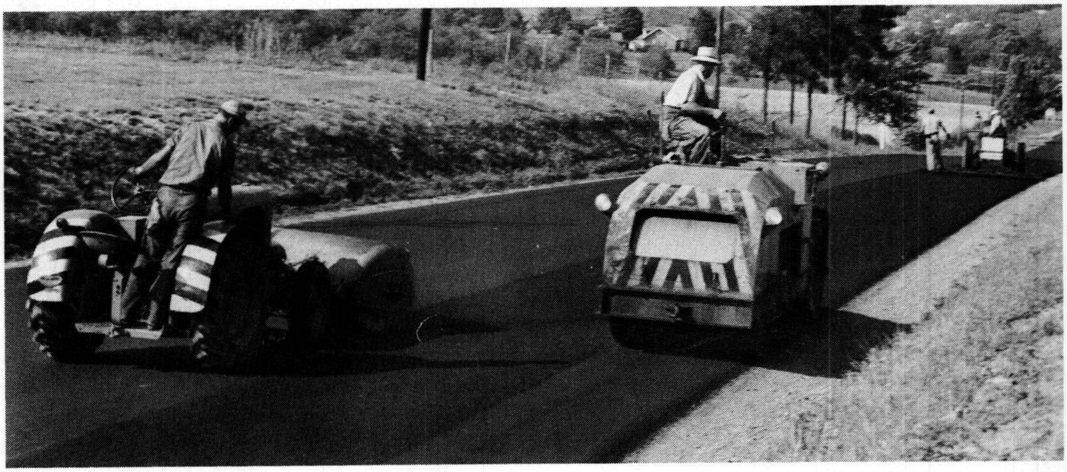

Figure 14.

road. This amount could be increased if a heating hopper were used.

The experimental mixes were never given a production test, but it is believed that a much higher output would be possible with little or no additional outlay in equipment. The spreading trucks could be loaded from dump trucks using a ramp provided near the job and the spreaders could be kept busy applying the material. From the experience gained during the experiments it is believed that four spreading trucks could apply at least 200 tons of plant-made deslicking mix per day as compared to 80 tons of rock

asphalt. The economical approach, of course, would be to accommodate the entire output of a plant and this would mean the utilization of more spreading trucks.

DISCUSSION OF EXPERIMENTAL MIXES

It will not be possible to evaluate the built-in test section placed in 1954 and 1955 until the 100 percent limestone control sections become polished. Therefore the sections containing polish-resistant coarse aggregate cannot be evaluated as yet. However, from the mixes placed in 1953 it has been learned that limestone mixes containing 20-25 percent polish-resistant fine aggregate (in this case, silica sand) will not consistently provide the necessary skid resistance.

Although the deslicking mixes have been down only six months it is believed that tentative conclusions can be formulated. The durability of fine sand deslicking mixes is probably the most essential part of their evaluation for it is known they will provide high frictional resistance as long as they remain on the road. Some speculation about the durability is possible based on some exploratory laboratory tests that were conducted. In outlining the laboratory tests it was reasoned that the factors contributing to the abrasion loss of a fine sand mix would be (1) lack of cohesion, and (2) loss of cohesion due to water action. Since mixes of this type are likely to be porous, the prevention of the loss of cohesion due to moisture was considered to be of paramount concern.

The lack of standard tests suitable for evaluating a thinly applied sand mix made the use of some improvised tests necessary. A boiling-stripping test was used to determine the resistance that the test blends would have to water, and a hand balling test was used to gage roughly the cohesion of the trial blends. While it is admitted that the boiling-stripping test has no positive correlation with field performance it nevertheless permits a comparative ranking of the experimental sand designs and rock asphalt. The laboratory tests pointed out that sands differ considerably in their inherent ability to resist stripping. Sand A, for instance, showed practically no ability to resist stripping even at temperatures of less than 150 deg F when used alone with asphalt. Upon adding hydrated lime (5 percent), however, it became impossible to strip the asphalt from Sand A even after boiling for 12 hours. At the end of this 12 hour boiling period the mix was still highly cohesive as gaged roughly by hand. However, Sand C exhibited good moisture resistance when used alone with asphalt, but was not benefited by the addition of the same amount of hydrated lime, and never achieved the moisture resistance of Sand A when hydrated lime was used.

Using the laboratory tests as a guide it was believed that plant-made deslicking mixes could be fabricated from sand, asphalt, and hydrated lime (if necessary) that would yield high cohesion and high moisture resistance as compared with rock asphalt. This conclusion was further validated by field observations made during the summer of 1955. Shortly after one of the plant-made deslicking mixes (Rte 11; Augusta Co) was installed, the pavement was subjected to the heavy and continuous rains that accompanied hurricanes Connie and Diane in August 1955. The severe moisture conditions resulted in the loss of some of the rock asphalt material which had been placed at about the same time but the experimental material remained intact.

For these reasons it is believed that a fine sand plant mix can be produced that will provide an adequate deslicking material which because of the use of local material will prove more economical in Virginia than rock asphalt. The field tests have shown that the approach is satisfactory but questions concerning specifications and test methods are still unanswered. The factors will be given attention as the next phase of the study.

CONCLUSIONS

In summary, the following points seem of significance in the over-all problem of providing skid resistant roads.

1. It has been demonstrated that pavements constructed of limestone become more slippery under traffic than those constructed from any other aggregate in Virginia. This is evident from the results of stopping distance measurements at 262 locations in the state, and, perhaps even more strikingly, from the number of skidding accidents in the three limestone districts contrasted with the number in the five districts containing little or no limestone.

2. In measuring skid resistance the stopping distance method is still recognized as the standard in Virginia. Recent improvements have enabled this test to be run swiftly and with very little inconvenience to traffic. The decelerometer method, however, may prove quite useful in securing a rough idea of skid resistance even more quickly than the stopping distance method; it has the added advantage of requiring a minimum of equipment and personnel.

1950 Experiments

3. Surface treatments utilizing a polish-resistant aggregate can provide good skid resistance. The tests show that slag was effective in this respect. It appears also that when 100 percent limestone aggregate is used, the size of aggregate (surface texture) will not significantly influence the skid resistance; limestone of any size will polish.
4. Rock asphalt (sandstone type) provides excellent deslicking material.

1955 Experiments

5. The "built-in" test sections including coarse and fine polish-resistant aggregate as a part of the asphaltic concrete aggregate blend and the machine-laid thin sand mix cannot be evaluated as yet.
6. Skid test results indicate that the addition of 20-25 percent silica sand to I-3 type mix will not consistently provide adequate skid resistance.
7. Laboratory tests indicate that deslicking mixes made from fine sand utilizing penetration grades of asphalt can be designed with high cohesion and high moisture resistance.
8. Field tests indicate that the plant-made deslicking mixes applied at rates of 10-15 psy can be expected to provide a suitable and economical method of eliminating slipperiness.

FURTHER RESEARCH

As shown in this report, Virginia has been working on the problem of providing skid resistant surfaces for over six years. During four of these six years some remedy has been tried in the field either as an experiment or as a part of the specifications. Even so only a few of the possible solutions have been tried and there are others that could be investigated.

To gain an insight into the practicality of some of the other potential remedies for slippery pavements, further field testing is planned. This will include:

1. The testing of blends of silica sand and limestone that will yield a finer (sand asphalt type) surface texture than the presently used I-3 asphaltic concrete.
2. The testing of sands coarser than those used experimentally for deslicking purposes. In addition, a more detailed investigation of the decelerometer method, with statistical analysis of data, will be undertaken.

Other areas remain, although Virginia has no immediate plans for their exploration. These include:

1. More detailed study of the effects on slipperiness of such factors as grade, tire tread and composition, pavement temperature, and the presence of films of oil, dust, or excess bitumen.
2. Correlation of work done in different states, aiming at the development of a standard method of test so that fairly accurate comparison of results may be made.

ACKNOWLEDGEMENTS

Many people have made contributions toward the solution of the problem of providing skid resistant roads in Virginia. Research personnel have constituted only a small part of this total effort. It is only proper, then, that the contributions of the many others besides the authors of this paper be recognized. The list of those who have assisted significantly with the over-all problem would include so very many names, however, that we have taken the liberty of speaking in general terms.

Therefore we would like to thank the field forces who have assisted in the running of

skid tests and the placing of the test sections. Also the cooperation of those in the Maintenance and Testing Divisions is greatly appreciated. We are grateful to many outside of the Department including the asphalt paving contractors who have been instrumental in assisting us in carrying out our field testing program. In addition we want to thank the Traffic and Planning Division for securing accident data for us. It has been encouraging to work with all of these people on a problem so vitally related to the safety and comfort of the traveling public.

The assistance of J. L. Eades, Geologist, Virginia Council of Highway Investigation and Research; Dr. Robert S. Young, Geologist, Virginia Geological Survey; and Dr. Byron Cooper, Head, School of Geology, Virginia Polytechnic Institute, in preparing the description of the types of limestone in Virginia (included in Appendix) is also greatly appreciated.

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Appendix

Notes on Virginia Geology and Aggregate Sources

Virginia may be divided into the following physiographic provinces, proceeding westward from the Atlantic Coast: the Coastal Plains, the Piedmont, the Blue Ridge, and the Appalachian Valley and Ridge.

The Coastal Plains lie between the seacoast and the Fall Zone and embrace all of Tidewater Virginia. This province is underlain chiefly by unconsolidated beds of gravel, sand, clay and marl. Bedrock is not generally encountered except at great depths. The only aggregates produced in the Coastal Plains are natural sands and gravels, the largest deposits occurring near the major streams and along the Fall Zone. Unlike some glaciated gravels found in other states, the gravels in Virginia are completely devoid of limestone.

The Piedmont province lies between the Fall Zone and the base of the Blue Ridge Mountains. The Blue Ridge province extends southwestward across the state between the Piedmont and the Appalachian Valley. Many of the same rock types underlie both these provinces: granites, gneisses, schists, basaltic lava flows, other igneous and metamorphic rocks of pre-Cambrian age, and folded sediments of Cambrian age. In addition, in the Piedmont may be found some Ordovician sediments, post-Ordovician granites, and Triassic sedimentary rocks with numerous igneous intrusives. The principal road construction aggregates produced in the Piedmont are igneous in origin: granites, gneisses, greenstones, and diabases (more commonly referred to as trap rocks).

The Appalachian Valley and Ridge province is rather narrow in its northern portion, lying between the Blue Ridge Mountains and the West Virginia border, but farther to the south it widens, covering the entire section known as Southwest Virginia all the way to the Kentucky border. Referring to Figure 1 found in Part I of this paper, it should be noted that this province covers practically all of Highway Districts 1, 2, and 8, the

TABLE A

SUMMARY OF SKID RESISTANCE DATA BY INDIVIDUAL AGGREGATE SOURCES

All Tests Made from 40 mph on Wet Pavement

Source of Aggregate	Type of Aggregate and Geological Formation	Type Tires	No of Sites Tested	Avg Distance to stop (feet)	Range of Distances to stop (feet)
Elkton Lime Co	Interbedded Limestone	A	4	142	
Elkton, Virginia	and Dolomite	B	3	103	
		C	0	-	
	Beekmantown (upper half)	D	0	-	
		All	7	125	100 - 157
Old Harrisonburg	Black Limestone	A	1	124	
Quarry, Harrisonburg,		B	6	126	
Virginia	Beekmantown	C	0	-	
		D	0	-	
		All	7	126	102 - 140
Stuart M Perry	Limestone	A	3	133	
Winchester, Va		B	0	-	
	Conococheague	C	0	-	
		D	4	114	
		All	7	122	113 - 140
Barger Bros Quarry	Black Limestone	A	7	132	
Lexington, Virginia		B	0	-	
	Whitesburg	C	12	111	
		D	7	133	
		All	26	123	95 - 148
Mundy's Rockingham	Brecciated Dolomite	A	3	128	
Quarry, Singers Glen,		B	2	116	
Virginia	Elbrook	C	0	-	
		D	6	125	
		All	11	124	110 - 137
Fred K Betts Quarry	Limestone	A	11	143	
Harrisonburg, Virginia		B	1	90	
		C	1	110	
	Beekmantown	D	18	118	
		All	31	126	90 - 157
Liberty Limestone	Dolomite	A	4	137	
Buchanan, Va and	Shady - Tomstown	B	7	139	
James River Hydrate	Limestone	C	3	119	
Indian Rock, Va	Holston, Lenoir and	D	1	130	
	Mosheim	All	15	134	105 - 169
Rockydale Stone Co	Dolomite	A	9	136	
Roanoke, Virginia		B	6	136	
	Rome Formation	C	0	-	
		D	7	126	
		All	22	133	99 - 162
Radford Limestone	Dolomite	A	0	-	
Radford, Virginia		B	0	-	
	Beekmantown	C	0	-	
		D	7	118	
		All	7	118	109 - 152
Blue Ridge Stone Co	Calcareous Shale -	A	7	132	
Blue Ridge, Va	Dolomite	B	2	112	
		C	0	-	
	Thinly Bedded - Elbrook	D	3	127	
		All	12	128	108 - 159
Holston River Quarries	Limestone	A	0	-	
Marion, Virginia		B	9	156	
	Mosheim	C	0	-	
		D	0	-	
		All	9	156	133 - 169
American Limestone Co	Limestone	A	0	-	
Kingsport, Tenn		B	5	133	
	Beekmantown	C	0	-	
		D	0	-	
		All	5	133	116 - 159
Millbrook Quarry	Limestone Con-	A	1	152	
Broad Run	glomerate	B	0	-	
		C	0	-	
		D	5	123	
		All	6	128	115 - 152
Bryan Rock and Sand	Silica Sand and Gravel	A	0	-	
Company		B	3	96	
Petersburg, Va		C	0	-	
		D	0	-	
		All	3	96	87 - 102

TABLE A (Continued)

Source of Aggregate	Type of Aggregate and Geological Formation	Type Tires	No of Sites Tested	Avg Distance to stop (feet)	Range of Distances to stop (feet)
Friend and Company	Silica Sand and Gravel	A	0	-	
Petersburg, Va and	Biotite Granite - Fine	B	2	98	
Trego Stone Co	Grained, Even-Textured	C	0	-	
Skippers, Virginia		D	0	-	
		All	2	98	93 - 103
R G Pope	Dolomite	A	0	-	
Bristol, Va		B	4	156	
	Conococheague	C	3	149	
		D	0	-	
		All	7	153	141 - 164
Dominion Limestone	Limestone	A	0	-	
Strasburg, Virginia		B	0	-	
	Mosheim	C	0	-	
		D	5	160	
		All	5	160	139 - 180
Kentucky Virginia Stone	Soft - Fine Grained	A	0	-	
Company	Limestone	B	5	133	
Gibson Station, Va		C	0	-	
	Lowville	D	0	-	
		All	5	133	113 - 155
Verona Quarry	Limestone - Interbedded	A	2	159	
Verona, Va	Limestone and Dolomite	B	0	-	
		C	0	-	
	Mosheim and Beekmantown	D	0	-	
		All	2	159	159
Riverton Limestone Co	Limestone	A	4	135	
Riverton, Virginia		B	0	-	
	Lenoir - Mosheim -	C	0	-	
	Beekmantown	D	0	-	
		All	4	135	130 - 137
Mundy's Shenandoah	Black Limestone	A	2	144	
Quarry		B	0	-	
Flat Rock, Virginia	Athens	C	0	-	
		D	0	-	
		All	2	144	140 - 148
Superior Stone Co	Quartz-Monzonite -	A	9	117	
Red Hill, Virginia	Gneiss	B	0	-	
		C	7	106	
		D	0	-	
		All	16	112	90 - 137
Greystone Granite Quarry	Shelton Granite Gneiss	A	0	-	
Shelton, North Carolina		B	6	90	
		C	0	-	
		D	0	-	
		All	6	90	79 - 109
State Quarry	Granite	A	0	-	
Horse Pasture, Va		B	1	91	
		C	0	-	
		D	0	-	
		All	1	91	91
Fairfax Stone Co	Diabase	A	2	134	
Centerville, Va	Grayish-black,	B	0	-	
	Medium grained	C	0	-	
		D	3	119	
		All	5	125	116 - 148
Arlington Stone Co	Diabase	A	5	130	
Herndon, Virginia		B	0	-	
		C	0	-	
		D	3	113	
		All	8	124	108 - 155

Bristol, Salem and Staunton Districts. This province is underlain by folded Paleozoic strata which include important deposits of limestone and dolomite, along with alternating layers of sandstone and shale in the mountains. Road aggregates produced in these areas are confined at present almost entirely to limestones and dolomites.

Table A includes such information as could be located regarding the geological formations encountered in the various quarries from which the aggregates for pavements included in the skid testing program were produced. The formations may be described as follows:

1. The Beekmantown formation, as found in the quarry of the Elkton Lime Company,

is composed of interbedded limestone and dolomite; a composite sample contained 57. 28 percent calcium carbonate, 37. 01 percent magnesium carbonate and 3. 52 percent silica. The Fred K. Betts quarry is in the upper part of the Beekmantown, and usually contains over 90 percent calcium carbonate. At Verona, part of the quarry was in the Beekmantown, both limestone and dolomite; the limestone was compact, dark gray to black in color, and contained 69. 51 percent calcium carbonate, 9. 63 percent magnesium carbonate, and 15. 59 percent silica, while the dolomite was fine, gray, and cherty, containing 56. 80 percent calcium carbonate, 38. 16 percent magnesium carbonate, and 2. 35 percent silica. At Riverton, the Beekmantown is a light gray limestone with 89. 10 percent calcium carbonate, 7. 69 percent magnesium carbonate, and only 1. 42 percent silica. At Radford this formation is light to medium gray dolomite with a 60-foot bed of black at the top; analysis showed 50. 65 percent calcium carbonate, 37. 80 percent magnesium carbonate, and 7. 30 percent silica.

2. The Conococheague formation exposed in the Stuart M. Perry quarry has banded limestone, magnesium limestone and thin layers of dolomite and sandstone; an average of two samples showed 68. 36 percent calcium carbonate, 16. 10 percent magnesium carbonate and 14. 10 percent silica.

3. No data were found on the formation in the Barger quarry at Lexington but analysis of a sample of the Whitesburg taken nearby showed 96. 71 percent calcium carbonate, 0. 09 percent magnesium carbonate, and 1. 62 percent silica.

4. The Elbrook formation at Mundy's quarry near Singers Glen was reported as a light to dark gray, mealy weathering brecciated dolomite; a sample of an 80-foot face showed 55. 36 percent calcium carbonate, 43. 12 percent magnesium carbonate, and 1. 56 percent silica. At Blue Ridge, the Elbrook is a dark gray, very fine grained, compact magnesium limestone; analysis: 58. 57 percent calcium carbonate, 26. 46 percent magnesium carbonate, and 9. 16 percent silica.

5. The Rome formation at Rockydale quarry is medium bedded, fine grained, dark gray dolomite; an analysis showed 52. 80 percent calcium carbonate, 40. 85 percent magnesium carbonate, and 1. 68 percent silica.

6. The Mosheim formation is a high calcium carbonate limestone at all three locations where analyses were available, and is described as dove gray and compact. At the Dominion quarry it is about 98 percent calcium carbonate and is found in thick beds. At Verona, the quarry has a cherty layer at the top, but contains 92. 61 percent calcium carbonate, 3. 79 percent magnesium carbonate, and 2. 36 percent silica. At Riverton, the Mosheim is thick bedded with 97 percent calcium carbonate.

No analyses from any of the other sources listed in Table A are available. However, the following brief descriptions have been found for individual sources, both in and out of the limestone areas:

1. Kentucky-Virginia Stone Company: a soft fine grained limestone of the Lowville

2. Riverton Limestone Company: the Lenoir formation (one of three known to be worked here), a dark gray, medium grained limestone.

3. Bryan Rock and Sand Company and Friend and Company: sub-angular to moderately rounded quartz and quartzite gravel, with varying percentages of feldspar.

4. Superior Stone Company, Red Hill quarry: an even granular to porphyritic, medium grained quartz-monzonite gneiss composed essentially of feldspar, quartz and biotite.

5. Greyston Granite Quarry at Shelton, North Carolina: a foliated granite gneiss of nearly uniform mineral content with microcline as its principal feldspar constituent along with quartz and muscovite.

6. Arlington Stone Company: and Fair-

TABLE B

TABLE OF AGGREGATE GRADATIONS SPECIFIED FOR VARIOUS TYPES OF BITUMINOUS CONCRETE

		Total Percent Passing									
Specification Year	Type	1	¾"	½	⅜'	No 4	No 10	No 40	No 80	No 200	% A C
1947 and 54	F-1				100	85-100	65-95	20-45	5-30	2-12	5 5-9 5
1947 and 54	I-3			100	80-100	50-70	35-50	10-25	3-15	2-10	5 5-8 5
1947 and 54	H-2	100	95-100		60-80	40-60	20-40		3-10		4 5-8 0
1938 Suppl	B-3			100	95-100	45-65	20-35		0-7 Approx		

fax Stone Company: a grayish black, fine grained diabase with about 48 percent plagioclase feldspar and 41 percent augite, uniform in texture, fresh, and very tough.

From the apparent variation in texture and chemical analysis within the same formations, and often within the same quarry, it seems evident that no insight can be gained into the skid resistance of a given aggregate from the analysis of one or two samples. Since the tests on practically all roads in the limestone area seem to indicate significantly longer stopping distances than those on roads in which gravel or granite have been used, it may be inferred that all Virginia limestones, no matter how dolomitic, tend to become slippery.

Discussion

E. A. WHITEHURST, Director, Tennessee Highway Research Program, The University of Tennessee — This report of studies of pavement slipperiness in Virginia has been of considerable interest to the Tennessee Highway Research Program, particularly inasmuch as the conclusion concerning the effect of limestone aggregates upon pavement slipperiness parallels so closely the one drawn and previously reported by this organization (1). Since the aggregates prevailing in Western Virginia are not appreciably different from many of those prevailing in Eastern Tennessee, their performances in pavements might be expected to be quite similar. It is indeed gratifying that independent studies have found this to be the case.

In the paper referred to above results of two types of pavement slipperiness tests were reported, those involving the actual measurement of stopping distances and those involving the measurement of the pavement coefficient of friction through the use of a skid trailer. During the past year and one-half supplementary tests have been made involving the use of the Tapley Decelerometer, the same instrument referred to by the

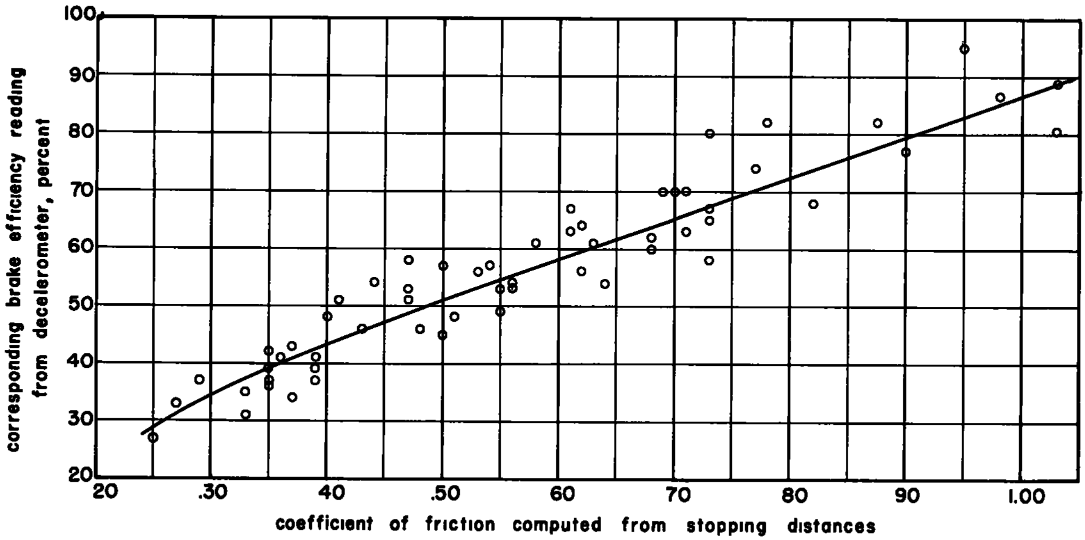

Figure A. Coefficients of friction versus decelerometer readings on same test run.

authors. The following data are offered to supplement their material concerning the use of this device.

It has been our experience that decelerometer results have relatively poor reproducibility if the test vehicle is permitted to decelerate through a large portion of the range between its original speed and zero. Best results are obtained when the wheels of the test vehicle are suddenly locked and then released as soon as the skid has started. Under these conditions reproducibility is very good indeed.

Tests involving both the stopping-distance method and the decelerometer method have been performed on 20 different pavement sections. Efforts were made to run the

tests from initial speeds of 10, 20, 30 and 40 mph. It soon became apparent, however, that the reproducibility of decelerometer reading for tests conducted from an initial speed of 10 mph was very poor. Such tests have, therefore, been discontinued.

The results of the tests performed on the 20 sections, from initial speeds of 20, 30 and 40 mph are shown on Figure A. This figure has been plotted on a scale similar to that used by the authors in their Figure 7. The degree of scatter appears to be about the same. The slope of the curve, however, is considerably different, the slope for the work in Tennessee being about 0.46, while that representing the tests made in Virginia is approximately 0.88. It is suspected that this difference may be due, at least in large part, to the differences between the tires used in the tests. The authors state that they used only tires having good treads. The work in Tennessee was done with tires which had been carefully capped to have smooth treads. There also seems to be a reversal of curvature between the data collected in Tennessee and that collected in Virginia.

In spite of these differences, however, it is believed that the results of tests in Tennessee complement those obtained in the Virginia tests and that the potential usefulness of the Tapley Decelerometer or some similar device as a quick measure of relative pavement slipperiness is clearly indicated. The differences between the two sets of data, however, equally clearly suggest the importance of a rather high degree of standardization of test procedures if results obtained by one organization are to be compared with those obtained by another.

REFERENCE

1. "Pavement Slipperiness in Tennessee," E. A. Whitehurst and W. A. Goodwin. Proceedings, HRB, Vol 34 (1955).