HIGHWAY RESEARCH RECORD

Number 341

Aggregates and Skid Resistance of Bituminous Pavements

7 Reports

Subject Areas

31 Bituminous Materials and Mixes
33 Construction

HIGHWAY RESEARCH BOARD

DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

WASHINGTON, D.C. 1970

ISBN 0-309-01846-3

Price: $1.60

Available from

Highway Research Board
National Academy of Sciences
2101 Constitution Avenue
Washington, D.C. 20418

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Foreword

The frictional resistance of pavements is a matter of growing concern because of the increasing requirements imposed by high volumes of traffic and the rapid wear and polish of pavement surfaces effected by this traffic. It is generally recognized that the aggregate exposed on the pavement's surface, particularly the coarse aggregate, is the major determinant of the skid resistance provided by the surface. A conference session was held during the 49th Annual Meeting to discuss these problems, and the papers in this RECORD were presented at that session. The objective of the conference session was to focus attention on the role of the aggregate in providing either skid-resistant or slippery pavements.

Sherwood and Mahone offer guidelines for the selection of carbonate aggregates based on the content of acid-insoluble residue. The discussion of this paper by Nichols includes data from circular-track traffic simulation tests that confirm the usefulness of the test for acid-insoluble constituents as an indicator of potential polish susceptibility.

Smith describes Maryland's circular-track simulator that is now being used to investigate the polishing proclivities of aggregates alone. A selected overview of European experience is given by Olsen. Gramling gives a progress report on Pennsylvania's extensive series of field test strips that have been constructed to evaluate the aggregate parameters controlling skid resistance. Serafin's description of Michigan's experience includes a number of interesting and informative observations on the construction and performance of skid-resistant bituminous surfaces.

The investigation reported by Rose, Hankins, and Gallaway relates the macrotexture of road surfaces to skid numbers and skid gradients, but the findings indicate that the microtexture of aggregate particles should also be considered. Preus describes the startling increase in wheelpath wear that has occurred on both bituminous and portland-cement concrete in Minnesota and that has been associated with the increased use of studded tires. Effects of this wear on skid resistance are not clear at this time.

—James M. Rice

Contents

Addendum to Highway Research Record 341

The Board inadvertently published the wrong committee sponsorship of the papers published in Highway Research Record 341. Discussions by F. P. Nichols, Jr., and Glenn Balmer of the paper, "Predetermining the Polish Resistance of Limestone Aggregates, by W. Cullen Sherwood and David C. Mahone, pages 1-10, which had been submitted in accordance with the Board's regulations and approved for publication, were omitted. This Addendum contains these materials and should be inserted in your issue of Highway Research Record 341.

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Discussion

F. P. NICHOLS, JR., National Crushed Stone Association—The paper by Sherwood and Mahone is a significant step in the direction of better understanding of the role played by aggregate mineralogy in the maintenance of adequate skid resistance. My own earliest work in this area is cited among the authors' references, and probably influenced the conclusion that, in the words of Sherwood and Mahone, "most of the limestones quarried in Virginia were unsuitable for use in the surfaces of high speed-high traffic volume pavements." Thus, it was decided that practically all limestones should be banned from use in pavement surfaces.

The references also include a 1960 article from the Crushed Stone Journal describing some of the earlier work at the National Crushed Stone Association. It was known then that not all limestones polish badly and that many which tend to can still be used in surface mixes when blended with more polish-resistant types. A later paper published in 1965 (13) described further work done along these lines in the NCSA's laboratory. The following discussion is offered to summarize the findings of a still later study designed specifically to permit the prediction of potential slipperiness of selected aggregates and blends of aggregates in bituminous mixtures. In this later study, comparisons could be made between aggregates based on the same mix gradations and test methods, an advantage not available to Gray and Renninger in reporting on the earlier work.

NCSA Test Track and Testing Procedure

The method of test used by NCSA for many years involves a 14-ft diameter track, 18 in. wide, that can accommodate as many as 20 test sections at a time. Severe traffic action is simulated by the continuous passage of a bus wheel loaded to about 2,000 lb on a single pneumatic tire. The details of the track and construction of the test sections have been described elsewhere (14).

The mixtures being tested are first subjected to wear to remove surface asphalt and to expose the aggregate particles. After this, the track is thoroughly cleaned, the initial slipperiness test is run, and the polishing action is begun. Slipperiness is usually measured after 2.5, 5, 10, 15, 25, 40, and 50 thousand wheel passes with a British portable tester in essential accordance with ASTM Standard Method E 303.

Research Plan

A plan formulated in May 1968 has been quite closely followed, though extended somewhat. The plan stipulated that an aggregate grading with ½-in. top size particles would meet both the appropriate ASTM specification and others commonly used by many state highway departments and also would combine durability, smooth riding quality, and resistance to hydroplaning. This grading, subsequently referred to as grading A, was held firm for all the mixtures tested under phases 1 and 2 of the plan. In phase 3 a similar test program was conducted on hot-mix seal treatments, one with the ⅜ in. open grading referred to as grading B, the other with a few plus ⅜ in. particles and the even, more open grading C. The 3 gradings are as follows:

Sieve	Grading A	Grading B	Grading C
½ in.	100		100
⅜ in.	85	100	95
No. 4	62	75	50
No. 8	48	25	15
No. 16		7	3
No. 30	26	—	—
No. 200	5	1±	—

Tests on Mixtures Containing Only Carbonate Aggregates

Unlike most studies, particularly that reported by Sherwood and Mahone, the current NCSA study made practically all aggregate identification tests on samples from the same batch of aggregate as was used in the bituminous concrete placed in the test track. This has not afforded a perfect correlation between skid resistance and either the total or the specific size fraction of insoluble residue, but it has made it clear that general relationships do exist, as will be shown.

The plan recognized that in ½ in. top size mixes the character of the largest particles exerts the most influence on the polish susceptibility of the surface. Therefore, the acid solubility test was conducted only on the plus No. 8 fraction of the grading. The method used to obtain and analyze the insoluble portion was essentially that described in an earlier paper (13): After the soluble carbonates were completely digested in diluted HCl, the residue was filtered, washed, dried, weighed, and then subjected to wet sieve analysis. No effort was spent on hydrometer analysis because it had been concluded earlier that the character of insolubles finer than 200 mesh was unlikely to affect polish resistance significantly.

A total of 24 carbonate rock samples were so tested. Of these, 22 were also subjected to the following more detailed analysis. A portion was ground in the laboratory ball mill until all would pass a 50 mesh, and the ground material was split into 2 parts; one was tested for maximum specific gravity, and the other by X-ray diffraction to detect the presence of the dolomite mineral.

Results of Test Program

Table 7 gives the findings on all 3 mix types and all 24 carbonate rocks.

Emphatic warning must be given that the BPT skid numbers obtained with NCSA's tester are relative to each other only. They may be higher or lower than BPT numbers that might have been obtained by other standard testers. Variables between test track installations may be present, but these were minimized by adjusting the tester each time to check as closely as possible with previous average readings on 4 standard surfaces. A roundabout correlation was once made that indicated that an NCSA skid number of 48 was roughly equivalent to the critical stopping distance number of 40 cited by

TABLE 7

TEST RESULTS OF ALL-CARBONATE AGGREGATES, NO BLENDS, AFTER 50,000 WHEEL PASSES

Date[a]	Aggregate	BPT Skid Number						Maximum Specific Gravity	Dolomite[c] (percent)	Acid Insolubles in -½ in. + No. 8	
		Grading A ½-in. Dense		Grading B ⅜-in. Open		Grading C[b] ½-in. Very Open					
		Original	Final	Original	Final	Original	Final			All +200	Sand
Aug. 1968	Penn. 1	73	70	78	68	80	67	2.72	None	49	34
Aug. 1968	Tenn. 2	74	69	81	69	—	—	2.76	Trace	32	27
Aug. 1968	Va. 3	69	58	76	58	76	58	2.76	Trace	30	18
Jan. 1969	N. Y. 2	72	57	—	—	—	—	2.83	Major	15	13
Jan. 1969	N. Y. 3	72	56	—	—	—	—	2.84	Major	17	11
Aug. 1968	N. Y. 1	64	56	—	—	—	—	2.71	Minor	36	5
Aug. 1968	Md. 1	65	54	72	56	73	48	2.74	20	9	7
July 1969	Me. 1	73	51	—	—	71	51	—	—	39[d]	12[d]
Aug. 1968	Tenn. 1	60	49	—	—	—	—	2.84	Major	8	4
Aug. 1968	Ga. 1	69	47	—	—	—	—	2.71	Trace	2	2
Jan. 1969	Penn. 3	62	47	—	—	—	—	2.84	Major	8	4
Aug. 1968	Tenn. 3	56	46	62	41	—	—	2.85	Major	4	2
Aug. 1968	Va. 2	58	45	—	—	—	—	2.82	Major	5	2
Aug. 1968	Va. 1(old)	59	44	—	—	—	—	2.85	Major	3	2
July 1969	Ill. 1	65	44	—	—	64	48	—	—	24	2
Aug. 1968	Penn. 2	55	43	—	—	—	—	2.82	40	2	1
Jan. 1969	Penn. 5	66	43	—	—	—	—	2.87	Major	3	1
Nov. 1968	N. Y. 4	65	43	—	—	—	—	2.84	Major	8	4
Jan. 1969	W. Va. 1	70	42	—	—	—	—	2.72	Minor	11	9
Jan. 1969	Penn. 6	64	42	—	—	—	—	2.85	40+	1	1
Feb. 1969	S. C. 2	71	41	—	—	—	—	2.72	None	3	3
Jan. 1969	Penn. 7	61	38	—	—	—	—	2.76	20	3	2
Aug. 1968	Va. 1(new)	60	37	—	—	—	—	2.74	Minor	1	1
Jan. 1969	Ohio 1	63	33	—	—	64	27	2.72	None	4	4

[a]Grading A only. [b]40,000 wheel passes. [c]Determined by X-ray diffraction. [d]Metamorphosed, insoluble grains very friable.

TABLE 8

TEST RESULTS OF NONCARBONATE AGGREGATES, NO BLENDS, AFTER 50,000 WHEEL PASSES

Aggregate Source and Type	BPT Skid Number					
	Grading A ½-in. Dense		Grading B ⅜-in. Open		Grading C[a] ½-in. Very Open	
	Original	Final	Original	Final	Original	Final
Penn. 9, sandstone	82	65	—	—	—	—
Conn. 2, granitic rock	73	55	—	—	—	—
Penn. 8, traprock	72	55	—	—	—	—
S. C. 1, granitic rock	68	55	69	54	—	—
Conn. 1, traprock	69	55	—	—	—	—
Md. 4, slag	71	54	—	—	—	—
Penn. 4, slag	62	54	65	49	—	—
Md. 2, traprock	66	53	—	—	—	—
Foreign, synthetic aggregate	68	51	—	—	—	—
Md. 3, siliceous gravel crushed	65	50	65	52	57	45
Va. 5, granitic rock	—	—	—	—	69	52
Va. 4, traprock	—	—	65	49	60	45
Ala. 1, slag	—	—	—	—	69	48

[a]40,000 passes.

TABLE 9

TEST RESULTS OF GRADING A, ½-IN. DENSE MIXES WITH BLENDING AGGREGATES AFTER 50,000 WHEEL PASSES

Date	Aggregate		Final BPT Skid Number			Percent Blend
	Primary	Blending	Primary Aggregate Alone	Blending Aggregate Alone	BPT Value Blend	
Aug. 1968	Ga. 1		47			67(+4)
		S. C. 1, granitic rock		55	52	
Jan. 1969	Penn. 3		47			
		Penn. 9, sandstone		65	55	67(+4)
Aug. 1968	Tenn. 3		46			
		Va. 4, traprock		—	48	67(+4)
Jan. 1969	Penn. 5		43			
July 1969		Penn. 1, carbonate rock		70	61	67(+4)
July 1969		Penn. 1, carbonate rock		70	53	33(+4)
July 1969		Md. 3, siliceous gravel crushed		50	52	67(+4)
July 1969		Md. 3, siliceous gravel crushed		50	50	33(+4)
July 1969		Md. 4, slag		54	51	67(+4)
July 1969		Md. 4, slag		54	48	33(+4)
July 1969		Md. 4, slag		54	45	67(+4)[a]
Jan. 1969		Penn. 8, traprock		55	51	67(+4)
Feb. 1969	S. C. 2		41			
		Md. 3, siliceous gravel crushed		50	45	67(+4)
Aug. 1968	Va. 1(new)		37			
Jan. 1969		Penn. 1, carbonate rock		70	53	67(+4)
Jan. 1969		S. C. 1, granitic rock		55	45	67(+4)
Jan. 1969	Ohio 1		33			
July 1969		Penn. 1, carbonate rock		70	59	67(+4)
July 1969		Penn. 1, carbonate rock		70	47	33(+4)
July 1969		Md. 4, slag		54	46	67(+4)
July 1969		Md. 4, slag		54	38	33(+4)
July 1969		Md. 3, siliceous gravel crushed		50	45	67(+4)
July 1969		Md. 3, siliceous gravel crushed		50	43	33(+4)

[a]All blends with Penn. 5 except this one are modified, little or no primary aggregate larger than ⅜ in.

TABLE 10

TEST RESULTS OF GRADING C, ½-IN. VERY OPEN MIXES WITH BLENDING AGGREGATES AFTER 40,000 WHEEL PASSES

Aggregate[a]		Final BPT Skid Number		
Primary	Blending	Primary Aggregate Alone	Blending Aggregate Alone	BPT Value Blend
Penn. 5		42		
	Penn. 1, carbonate rock		67	61
	Va. 5, granitic rock		52	50
	Ala. 1, slag		48	47
	Md. 3, siliceous gravel crushed		46	48
Ohio 1		27		
	Penn. 1, carbonate rock		67	61
	Va. 5, granitic rock		52	52
	Md. 3, siliceous gravel crushed		46	48
	Ala. 1, slag		48	44

[a]The 50 percent of primary aggregate met simplified practice grading No. 9; the blending aggregate was slightly coarser, meeting No. 8, with the result that 88 percent of the plus No. 4 particles were from the more polish-resistant rock and 88 percent of the minus No. 4 were from the primary carbonate rock.

Sherwood and Mahone, but this relationship has never been confirmed. Further, any such relationship would no doubt be different for the different surface textures resulting from gradings A, B, and C. Perhaps the only statement that could be made with real confidence would be that, for a given gradation, the higher the BPT number is the higher the skid resistance will be.

Notwithstanding these statements, the relationships tabulated are believed to have real value in explaining the role of rock type and composition in the complex picture of skid resistance. Under each grading given in Table 1 is shown an original test value (obtained after initial wear and cleaning) and a final test value (obtained at the termination of the test, usually 50,000 wheel passes). The insoluble residues from the coarse aggregate particles are expressed as "all +200" and "sand," and are percentages by weight after being digested in HCl, washed, and sieved. Sand refers to the minus No. 8 and plus No. 200 material in the residue.

Table 8 gives the applicable test data obtained on the 13 noncarbonate aggregate mixes. Only one of these, the crushed siliceous gravel, is noted to have been tested in all 3 gradings.

Table 9 gives the effectiveness of blending various percentages of the more polish-resistant aggregates with 7 of the less polish-resistant carbonate aggregates in the plus No. 4 fraction of the grading A mix. Because this mix had only 38 percent plus 4, the substitution of ⅓ of this fraction was equivalent to roughly 13 percent of the total aggregate—⅔ to roughly 25 percent. Note that the figures in the columns "Primary Aggregate Alone" or "Blending Aggregate Alone" were transposed from Tables 7 or 8 as applicable.

Table 10 gives similar data for open-graded mixes B and C, indicating the feasibility in certain areas of using 50 percent of a more readily available carbonate aggregate in an adequately skid-resistant, open-graded mixture. The note to Table 10 explains how the effectiveness of the blending aggregate was increased by ensuring that it compose most of the larger particles in the mix.

Discussion of Results

Figure 5 shows the relationship between acid-insoluble, sand-sized constituents in the coarse fraction of all-carbonate mixtures and the final BPT number obtained after polishing. The line shown represents the least squares regression computed in the

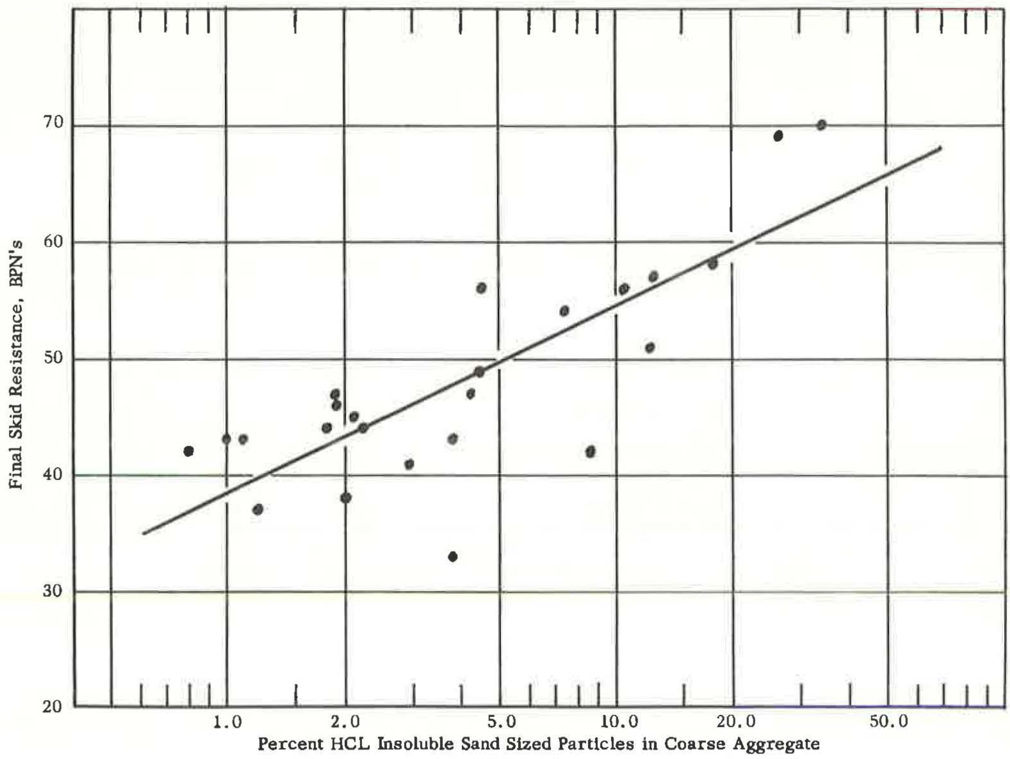

Figure 5. Relationship between final skid numbers and sand-sized insoluble particles in coarse aggregate after 50,000 wheel passes.

normal manner except that the x-values used, instead of being the actual percentages of insoluble material, were the logarithm of those percentages; these computations resulted in determining a standard error of estimate of 5.6 British portable tester numbers and a coefficient of correlation of 0.80. The actual plot, it may be noted, is on a semilogarithmic scale, so that the percentages of insoluble material are shown directly rather than as their logarithms.

This correlation is not as close as might be desired, and some anomalies are evident. However, it is obvious that the rocks that contain more than 5 percent sand-sized insolubles all produce mixes with final British portable tester numbers of more than 50—generally as good as or better than the mixes from slags or most of the harder rocks given in Table 8.

A similar relationship may be plotted on the basis of total insoluble, but more anomalies are noted in cases where the total insoluble figure is quite high but the sand size insoluble is low, e.g., Ill. 3 and W. Va. 1. These are limestones containing solid chert particles that are not digested at all by the acid solution.

The carbonate rocks with high percentages of insoluble sand-sized particles, and the one sandstone in the study (Penn. 9), are highly effective as blends with the purer carbonates to increase polish resistance of mixes, as may be noted from data given in Tables 9 and 10. No other aggregates tested approached the effectiveness of Penn. 1 as a blending aggregate, but in all the blends tested the addition of the more polish-resistant aggregate appreciably improved the skid resistance of the all-carbonate mixes made from the rocks with low percentages of insolubles.

It is interesting to note the distinct relationship between maximum specific gravity and percentage of dolomite for the 24 carbonate rocks given in Table 7. Those rocks with little or no dolomite all had maximum specific gravities of 2.76 or less. Those reported to have either a major percentage of dolomite or as much as 40 percent all had specific gravities of 2.82 or more. This seems to indicate that a fair estimate of the percentage of dolomite may be obtained from the much simpler test for maximum specific gravity without the necessity of X-ray diffraction analysis. In this connection, it appears that the NCSA data confirm those cited by Sherwood and Mahone to the effect that there is a lack of any trend between calcite dolomite percentages and skid resistance.

General Indications From NCSA Data

The studies conducted by NCSA since May 1968 seem to definitely indicate the following:

1. Many carbonate aggregates may be used with as great a degree of safety as many slags or so-called hard rocks in the design of skid resistant surfacing mixes.
2. The test for acid-insoluble constituents provides a good preliminary indication of the potential polish susceptibility of carbonate rock deposits used or being considered for use in the production of aggregates. The test should be performed on the coarse particles, and judgment should be used in deciding whether total insolubles or only sand-sized insolubles should be considered the best indicator in the case of particular rocks.
3. Careful blending of uniformly high friction aggregates is both practically and economically feasible in upgrading to an acceptable level the skid resistance of mixes containing from 50 to 87 percent relatively pure carbonate aggregates normally expected to be polish susceptible.

Acknowledgment

Appreciation is expressed to the G. and W. H. Corson Company for conducting the X-ray diffraction tests.

References

13. Gray, J. E., and Renninger, F. A. The Skid Resistant Properties of Carbonate Aggregates. Highway Research Record 120, 1965, pp. 18-34.
14. Goldbeck, A. T., and Gray, J. E. Skid Proofing of Asphaltic Concrete Pavement Surfaces. Crushed Stone Jour., National Crushed Stone Assn., March 1969.

GLENN BALMER, Federal Highway Administration—The authors' use of the acid-insoluble residue test for predetermining the skid-resistance characteristics of limestone aggregate shows application of a simple and pertinent procedure. They are to be congratulated for a fine paper.

It is significant to note in another paper (15) that the friction characteristics of portland cement concrete pavement with limestone in the surface course also increased with the increase in acid-insoluble residue content of the limestone. The test results are similar to those of Sherwood and Mahone.

Furthermore, the skid resistance of the pavement was increased by blending of aggregates that increased the siliceous particle content of the surface aggregates as Nichols has commented in his discussion.

The acid-insoluble residue determination is a convenient means of predetermining potential skid resistance of pavements and should be formulated as a standard procedure.

Reference

15. Laboratory Test Results of the Skid Resistance of Concrete. Jour. of Material, Sept. 1966.

Predetermining the Polish Resistance of Limestone Aggregates

W. CULLEN SHERWOOD, University of Virginia; and
DAVID C. MAHONE, Virginia Highway Research Council

Research data from a variety of sources have shown conclusively that aggregate type is a vital factor in pavement friction and that limestone aggregates tend to polish more readily than do other commonly used aggregates. It has also been established that significant differences in polish susceptibility exist among limestones and that these differences are related primarily to the noncarbonate or acid insoluble constituents in the rock. Since 1946, the Virginia Highway Research Council has accumulated a large number of pavement friction measurements and a great deal of information on aggregate characteristics. Based on these data it appears that a simple relationship exists between the total acid insoluble content of Virginia limestones and their polish resistance as evaluated by stopping distance numbers. On the basis of this relationship and observations on traffic volumes, tentative guidelines for the use of limestones in pavement surfaces are proposed. The essence of these guidelines involves the use of limestones with increasingly higher insoluble fractions in pavement surfaces carrying progressively greater traffic volumes. In this way it is thought that certain relatively impure limestones may be used in pavements that carry up to 10,000 vehicles per day. It is felt that, if selected limestones can be used in pavement surfaces, considerable economies will result because many states are either excluding or are considering exclusion of all limestones from pavement surfaces. It is recommended that other states carry out similar studies relating pavement friction to limestone aggregate characteristics, particularly the insoluble content. In this way the tentative guidelines suggested here may be refined and found useful in areas outside of Virginia.

•ONE OF THE most pressing problems in highway construction and maintenance is that of building in and maintaining adequate skid resistance in pavement surfaces carrying large volumes of high-speed traffic. This problem was recognized in Virginia as early as 1946 when the continuing program of skid resistance research began. Early research in this field brought to light several interesting facts: (a) In some areas of Virginia skidding accidents were significantly higher than in other areas of the state; (b) aggregate type had a significant effect on coefficient of friction values; and (c) the coarse aggregate in bituminous and the fine aggregate in concrete have the most effect on the friction properties of the respective pavements. The investigations reported here involved bituminous surfaces only. It was subsequently noted that limestones comprised the major source of aggregate in those portions of the state where rates of skidding accidents were high, and that the great majority of all of the slippery pavements measured statewide were surfaced with limestone aggregates. (The term "limestone" is used to include both limestone and dolomite rocks.)

Paper sponsored by Committee on Bituminous Aggregate Bases and presented at the 49th Annual Meeting.

Following these early efforts, a significant amount of skid test data was gathered with both the stopping distance method and a skid trailer. These data indicated that most of the limestones quarried in Virginia were unsuitable for use in the surfaces of high speed-high traffic volume pavements. Consequently, specifications were drawn up to exclude the use of limestones in the surface courses of Interstate and primary roads. Subsequently, it was found that 2 sources of impure limestone quarried from the Arch Marble formation in the Lynchburg area produce aggregates that provide relatively high skid resistance; therefore, the 2 Lynchburg quarries are permitted to provide aggregate for all pavement surfaces except those in the Interstate system.

Once it became apparent that most of Virginia's limestones did polish and become slippery, research aimed at remedial measures was undertaken. Blends of nonpolishing aggregate and limestones were used in surface courses on pavements in Virginia as early as 1955, and experiments with thin silica sand overlays were instigated by Dillard et al. (5) during the same year. These measures proved highly successful in increasing pavement friction and are now used extensively in those portions of Virginia where limestone is the major source of aggregate.

Although the majority of limestones tested tend to become slick when subjected to heavy traffic, they differ significantly in polish susceptibility. This difference was specifically noted by Shelburne and Sheppe (8), Nichols et al. (5), and Nichols (6) in Virginia, and several investigators in other states. Further research by Sherwood (9) and Gray and Renninger (4) showed that the amount and nature of the acid insoluble mineral grains contained in limestones were primarily responsible for their variable wearing characteristics. Goodwin (3) describes several other methods that are and have been used to pre-evaluate pavement materials for skid resistance.

Although some states and federal agencies have outlawed the use of limestones in heavily traveled road surfaces, no state or agency, to the writers' knowledge, has attempted to set forth specific standards or test procedures whereby limestones could be differentiated on the basis of polish susceptibility and specified for use accordingly.

The writers believe that, in the absence of a test procedure for predetermining the polish susceptibility of limestones, many states have and will continue to arbitrarily outlaw all aggregates of this type in surface courses. To safeguard against the waste that results from this decision, the writers feel it is essential that criteria be developed to enable pavement mix designers to classify limestone aggregates with regard to their skid-resistance properties.

Therefore, the purpose of this report is to propose acid insoluble tests and specification limits for this differentiation and to establish tentative guidelines for the use of limestones in selected pavement surfaces. Because this is the initial effort to establish such guidelines, those proposed should be considered tentative at best. However, the writers believe that the experience gained in Virginia may serve as a starting point; and if other states and federal agencies gather and make available data on limestone properties and related pavement friction, then the guidelines can be changed to reflect this additional input of information.

One final note: This paper does not pretend to report on a single study that was carefully designed and controlled. It is more a compilation of skid data accumulated in Virginia over a period of time by several investigators by several test methods. These results, in turn, have been correlated with acid insolubles from the limestones involved In some instances, careful control may have been lacking. However, it is the writers' belief that enough data representing a broad spectrum of pavement ages and conditions have been accumulated and carefully processed to permit establishment of some valid trends and conclusions.

SOURCES AND COMPOSITION OF LIMESTONE AGGREGATES IN VIRGINIA

Limestone is the primary source of aggregate in the valley and ridge physiographic province of western Virginia. This section encompasses roughly one-third of the state (Fig. 1). The carbonate rocks found here are both limestones and dolomites. They formed as part of the Appalachian geosynclinal sequence and range in age from Cambrian

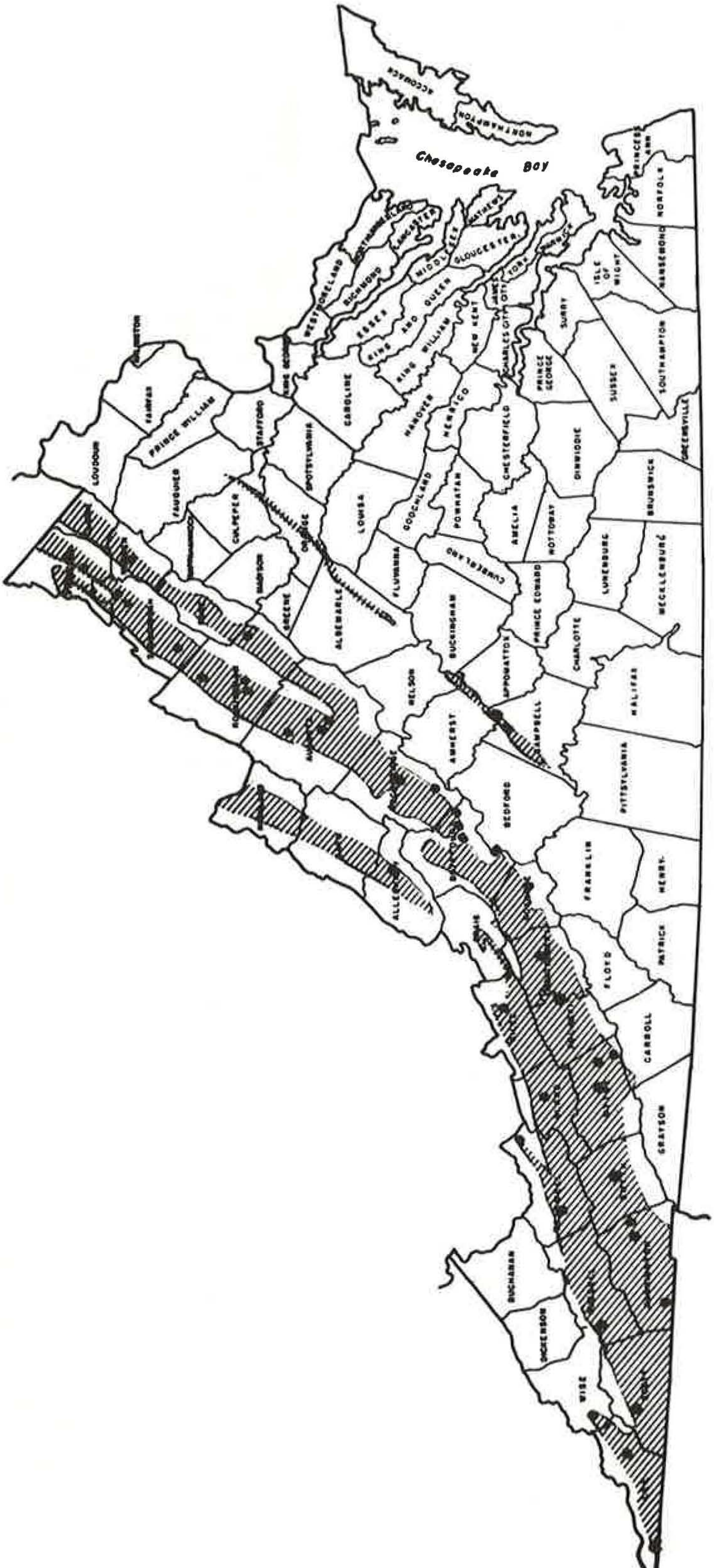

Figure 1. Limestone areas and quarry locations.

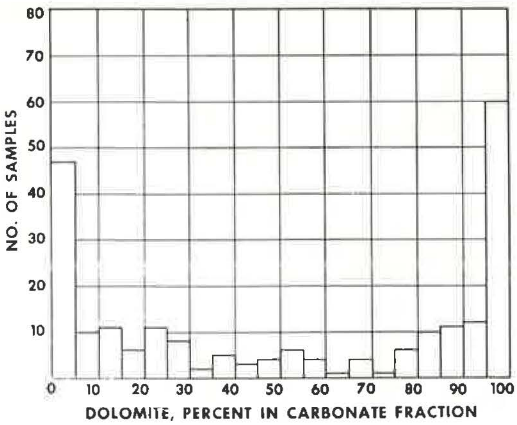

Figure 2. Distribution of carbonate compositions of 224 test samples.

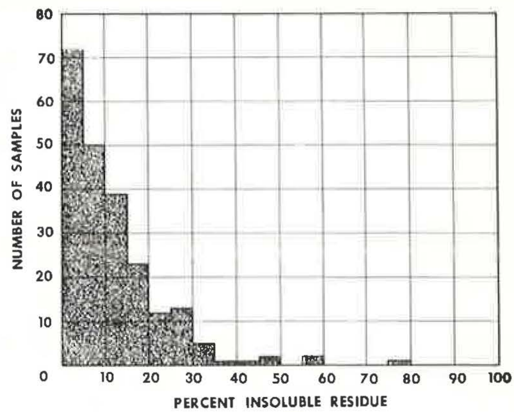

Figure 3. Distribution of insoluble residues determined for 224 test samples.

to Mississippian. Figures 2 and 3 show the general composition of carbonate aggregates quarried in the state (10). Individual rock samples tend to be either predominantly calcite or predominantly dolomite rather than mixtures of the 2 minerals (Fig. 2). For example, of the 224 samples processed, 47 were predominantly calcite and contained less than 5 percent dolomite, while 60 were predominantly dolomite and contained less than 5 percent calcite. The range of insoluble residues for the same group of 224 samples is shown in Figure 3. Clearly, limestone aggregates produced in Virginia tend to contain low percentages of insolubles, with more than 50 percent of the samples falling in the interval between 0 and 10 percent. On the other hand, the 3 highly impure limestones considered by Gray and Renninger (4) as having excellent skid-resistance properties contained from 36 to 48 percent insolubles. It would appear that these kinds of rocks are typical of the vast majority of Virginia limestone aggregates (Fig. 3).

In order to relate some of the available data on skid resistance to limestone composition, the present paper considers aggregates from 25 major limestone-producing quarries in Virginia. Table 1 gives the geologic formations in which the quarries are located and a brief description of the rock type or types quarried. Several quarries are producing stone from more than one geologic formation. For instance, the Augusta, Verona, and Holston River quarries produce both the high calcium New Market-Five Oaks limestone and the more dolomitic Beekmantown. Pounding Mill produces from at least 3 middle Ordovician formations.

SKID-RESISTANCE DATA AND ROCK PROPERTIES

During the past 23 years skid test data and coefficient of friction measurements have been compiled for Virginia's pavements by a variety of methods. This compilation includes a large amount of information on limestone and other pavement surfaces. Fortunately from the standpoint of safety but unfortunately from the research viewpoint, limestone pavements carrying relatively high traffic volumes have been covered by skid-resistant overlays. Also, as mentioned earlier, no new limestone surfaces are being constructed on primary routes. Consequently, it is now difficult to accurately evaluate the differences in the wearing characteristics of Virginia limestones, and this paper must be based to a large extent on past studies. However, most of the acid insoluble data are new as are the analyses of the relationship on the interdependency of skid resistance on quantity of acid insoluble material.

For the sake of uniformity, all of the skid test data have been converted to and reported as 40-mph stopping distance skid numbers (SDN_{40}). The term SDN is equal to

TABLE 1

GEOLOGY OF LIMESTONE QUARRY SOURCES

Age	Formation	Quarry	General Lithology
Ordovician	Edinburg	Barger, Mundy No. 2, Frazier	Dark gray to black, dense limestone with contrasting white calcite veins. Medium bedded but frequently shaley partings affect rock breakage and shape.
	Lowville	Kentucky-Virginia	Fine-grained, light gray limestone with mottled, reddish and fossiliferous beds, with minor shale and chert.
	Perry-Ward, Cove-Lincolnshire	Pounding Mill	Varied lithology composed of a series of interbedded fine limestones, coarse-grained fossiliferous limestones, and medium-grained limestones, with some chert nodules and shaley partings.
	New Market (Mosheim)-Five Oaks	Augusta, Verona, Chemstone, Swords Creek, Holston River	Very fine-grained, dove gray, high calcium limestone (calcilutit). Clear, coarsely crystalline calcite "eyes" are common.
	Beekmantown	Elkton, Riverton, Betts, Augusta, Belmont, Radford, Holston River, Pembroke, Virginian	Series of interbedded dolomites and limestones. Texture ranges from very fine to medium and color from very light to dark gray. Beds are thick and massive with some chert. Pembroke quarry is located in a high magnesium dolomite with less insoluble residue than normal Beekmantown.
Cambrian	Conococheague	Perry, Pendleton	Conococheague is quite variable. Rocks exposed are medium to thick bedded, medium-grained limestones, and dolomitic limestones. Color is dark gray with white calcite fracture fillings.
	Elbrook	Blue Ridge, Mundy No. 1	Elbrook is quite variable. At the Blue Ridge quarry, fine to medium grained, dark gray dolomites, and light gray shaley dolomites are most common. The Mundy quarry is located in massively bedded, highly fractured (brecciated) dolomites. Two distinct rock colors, one a pinkish tan and the other a medium gray, can be seen.
	Shady	Rockydale, Liberty	The Shady is a relatively pure dolomite ranging in color from medium gray to light tan and off-color white. Bedding is usually massive, and blends of white, recrystallized dolomite are common. Texture is fine to medium.
Unknown (probably early Paleozoic)	Arch Marble	Lynchburg	Metamorphosed, impure limestone. Rocks are medium to coarse grained with variable amounts of muscovite, biotite, and quartz. Bedding and foliation both affect rock fracture, yielding slabby to equidimensional particles.

the stopping distance coefficient of friction times 100 when the coefficient of friction is calculated as follows:

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

where

μ = coefficient of friction;
V = test velocity; and
S = stopping distance in feet.

The conversion curves for determining the SDN_{40} values from data of tests performed by methods other than the stopping distance car are taken from Dillard and Allen (2).

A compilation of the available data for skid tests made on wet pavements containing aggregates from the sources selected for this study is given in Tables 2 and 3. Table 2 gives a resume of the skid test data for each quarry source. The number of sections tested and the total number of tests for all sections are given, along with the range for all tests and the average for each quarry source. All of the sources studied show a wide range of stopping distance values, undoubtedly reflecting a wide range in age, traffic volume, and general pavement condition. All but a few produce extreme values considerably higher and lower than the minimum SDN_{40} of 40 observed in Virginia as desirable. Although there is no required minimum for new construction in Virginia, it has been the policy of the Virginia Department of Highways to resurface all primary, arterial, and Interstate highways that are found to have a SDN_{40} below 40.

TABLE 2

SKID TEST DATA FOR VIRGINIA LIMESTONES

Quarry	Number of Sections	Number of Tests	SDN_{40}[a] Range	SDN_{40}[a] Average	Rating
Lynchburg	3	47	47-59	53	Good
Blue Ridge	24	200	32-57	44	Fair
Elkton	11	78	31-52	44	
Mundy No. 1	13	50	32-47	43	
Betts	33	119	34-57	43	
Barger	34	177	29-54	42	Poor
Belmont	1	10	35-50	42	
Pendleton	4	66	35-52	42	
Perry	17	149	31-46	42	
Kentucky-Virginia	5	15	35-46	40	
Rockydale	28	159	29-52	40	
Liberty	19	86	31-49	40	
Riverton	8	55	29-46	40	
Radford	15	121	26-48	40	
Augusta	1	20	28-46	39	Unacceptable
Virginian	4	47	32-46	39	
Pounding Mill	7	83	26-46	38	
Mundy No. 2	4	33	32-43	38	
Holston River	12	54	36-46	37	
Frazier	2	27	31-43	36	
Swords Creek	1	25	21-43	34	Critical
Verona	2	6	34	34	
Pembroke	1	12	30-35	33	
Chemstone	12	75	19-39	31	

[a]Use of SDN (stopping distance number) conforms to the usage of Dillard and Allen (2).

TABLE 3

COMPOSITION AND COEFFICIENT OF FRICTION VALUES FOR VIRGINIA LIMESTONES

Quarry	SDN_{40}	Total Insolubles (percent)	Average Insolubles (percent)	Calcite-Dolomite Percent of Total Carbonate
Lynchburg	53	37	37	100-0
Blue Ridge	44	17	13	64-36
Elkton	44	10		60-40
Mundy No. 1	43	13		1-99
Betts	43	13		54-46
Barger	42	11	10	83-17
Belmont	42	12		63-35
Pendleton	42	5		5-95
Perry	42	12		54-46
Kentucky-Virginia	40	9		86-14
Rockydale	40	9		1-99
Liberty	40	10		1-99
Riverton	40	3[a]		57-43
Radford	40	9		3-97
Augusta	39	6	8	10-90
Virginian	39	8		2-98
Pounding Mill	38	8		82-18
Mundy No. 2	38	9		97-3
Holston River	37	8		10-90
Frazier	36	7		93-7
Swords Creek	34	3	5	95-2
Verona	34	7		
Pembroke	33	6		5-95
Chemstone	31	2		100-0

[a]Appears to be anomalously low.

TABLE 4

GRAIN SIZE OF INSOLUBLE RESIDUES FROM SELECTED QUARRIES

Quarry	SDN_{40}	Total Insolubles (percent)	Grain Size		
			Percent +270	Percent +140	Percent +10
Lynchburg	53	37	24	20	6
Blue Ridge	44	17	12	11	5
Mundy No. 1	43	13	5	4	2
Betts	43	13	1	1	1
Belmont	42	12	1	trace	trace

The average SDN should also be noted. Only the Lynchburg quarries are producing limestone that could be classified as high friction aggregates. Another group of 4 quarries might be considered as fair, and 9 quarries as poor. Average results from the remaining 10 quarries are unacceptable by Virginia standards. Values of 34 or less are rated as critical and indicate a distinct hazard to drivers at even moderate speeds. Because of the generally low average values and the presence of sites testing well below 40, the use of all limestones—except the Lynchburg material—has been banned in Virginia from primary, arterial, and Interstate surfaces.

Results of insoluble residue and carbonate mineral analyses are given in Table 3. Here it is clear that a direct relationship exists between insoluble residue percentage and aggregate-wearing characteristics as reflected by SDN_{40} values. Equally clear is the lack of any trend when calcite-dolomite percentages are compared with SDN_{40} values.

In a separate study, which is not yet complete, the grain size of the insoluble residue fraction is being measured and compared with aggregate-wear characteristics. As would be expected from the previously mentioned works of Sherwood (9) and Gray and Renninger (4), and as substantiated by Burnett, Gibson, and Kearney (1) in New York, the size distribution of the insoluble fraction affects pavement friction values. Preliminary results of this study are given in Table 4.

Although this relationship between coarseness of insolubles and coefficient of friction does appear to exist, it appears that an even better relationship may exist between total insolubles and friction coefficients. This appears to be especially true in the case of the Betts material, which contains 13 percent total insoluble materials but only 1 percent greater than the 270 mesh. Yet the SDN_{40} for surfaces made with Betts is relatively high at 43. Unquestionably the relationships involved here require further detailed study.

Efforts have been made to correlate overall rock texture (grain size) with polishing characteristics in a manner analogous to that used by Shupe and Lounsbury (11). Unfortunately, most of Virginia's limestones are quite similar in texture, and no obvious relationships could be ascertained. Studies of aggregates produced in other states may yield a more definite relationship between texture and polish characteristics and should be undertaken.

DISCUSSION OF RESULTS

In the classification of Virginia limestones in order that their relative skid resistance might be predetermined, 2 criteria are used. The most obvious method is the use of total insoluble residue percentages. Table 5 gives the average percentage of insolubles for each of the 5 groups for which ratings are given in Table 2. A relationship between coefficient of friction and insoluble residue appears to exist. So that a closer look could be taken

TABLE 5

RATING OF INSOLUBLE RESIDUES AND COEFFICIENT OF FRICTION VALUES

Rating	Average SDN_{40}	Average Insolubles (percent)
Good	53	37
Fair	44	13
Poor	41	9
Unacceptable	38	8
Critical	33	5

of this relationship, Figure 4 was constructed by plotting coefficient of friction versus insoluble residue for each quarry source. A fairly well-defined curvilinear relationship, similar to that obtained by Burnett et al. (1) for insolubles coarser than 200 mesh, is evident. It is thought that more detailed sampling and insoluble residue measurement for these and other limestone sources will clarify this relationship.

A second method of classifying the polish resistance of these groups of aggregates involves the relationship of skid numbers and geologic formation. The results of this treatment are given in Table 6. Average percentages of insolubles are also included, and they provide additional insight into this relationship. Geologic formations show varying degrees of polish resistance.

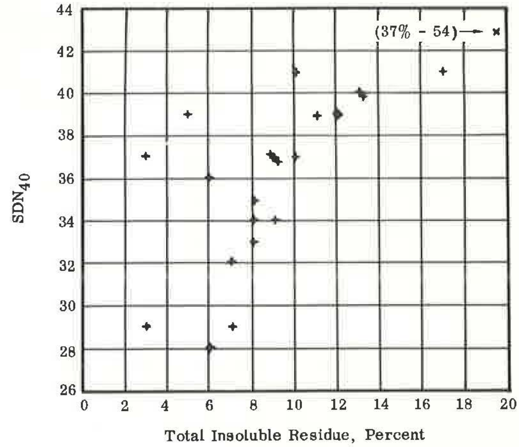

Figure 4. Effect of insoluble residue on polish resistance.

Further insight into the effect of the grain size of insolubles on skid numbers is also forthcoming. For example, data given in Table 6 show that the Conococheague and the Edinburg formations both contain 9 percent insolubles but have SDN_{40} values of 42 and 39 respectively. Based on other studies, it is well known that the major portion of the insolubles from the Edinburg are in the clay range, whereas the Conococheague contains considerable sand and silt. Insolubles from the New Market-Five Oaks limestones, which provide a very low SDN_{40} of 35, are very fine.

TENTATIVE GUIDELINES

The philosophy on which this paper is based is that the suggestions made here should serve as a base for further development rather than as precise, rigorous standards. Certainly, as data accumulate from other geographic and geologic areas and from within other agencies, these suggested limits on the use of limestone aggregates will be altered. With this in mind, we offer the following tentative guidelines for the use of limestones in pavement surfaces.

TABLE 6

COEFFICIENT OF FRICTION VALUES FOR AGGREGATES FROM SELECTED GEOLOGICAL FORMATIONS

Formation	Average Insolubles (percent)	SDN_{40}	
		Range	Average
Arch Marble	37	—[a]	53
Elbrook	15	43-44	44
Conococheague	9	42	42
Beekmantown	8	33-43	41
Shady	9	40	40
Lowville	9	40	40
Edinburg	9	36-42	39
Perry-Ward, Cove-Lincolnshire	8	38	38
New Market (Mosheim)-Five Oaks	5	31-37	35

[a]Pavements made with Arch Marble aggregates were not separated on the basis of quarry source when skid tested.

1. For Interstate and other heavily traveled roads with a traffic count of more than 10,000 average vehicles daily, no carbonate aggregates have been located in Virginia that would be suitable for unlimited use in the surfaces of these highways.

2. For roads with a traffic count of less than 10,000 average vehicles daily, limestones of the Arch Marble type quarried at Lynchburg with insolubles over 30 percent and preferably coarse grained are suitable.

3. For roads with a traffic count of less than 4,000 average vehicles daily, aggregates from the Elbrook formation (Table 6) or those classed as fair (Table 2) from Blue Ridge, Elkton, Mundy No. 1, and Betts quarries with insoluble residue minimum of 15 percent appear to hold promise.

4. For roads with a traffic count of less than 1,000 average vehicles daily, aggregates with insoluble residue minimum of 10 percent, including most of the sources classed as poor or above (Table 2) are suitable.

5. For roads with a traffic count of less than 150 average vehicles daily, any limestone that passes the normal physical tests should be acceptable.

SUGGESTIONS FOR FURTHER STUDY

The following suggestions are offered to other workers interested in the role of limestone aggregates in pavement surfaces. These may be particularly appropriate in states where considerable limestone is still exposed in pavements carrying high traffic volumes.

1. Survey pavement friction for specific regions where a variety of limestones have been used in the pavement surfaces and where accurate records on aggregate sources are available. In this way, high and low friction values may be related to specific quarry sources and geologic formations.

2. Conduct detailed sampling and laboratory analyses of sources providing limestone aggregates for the pavements surveyed. The analyses should include total insoluble residues, grain size, and mineralogy of insolubles and may include carbonate mineralogy and texture. (Methods used for determining total insoluble residues and calcite-dolomite ratios are outlined in the Appendix.) It will be particularly critical to determine the relative effect of fine versus coarse insolubles on skid resistance.

3. Correlate limestone pavement friction measurements with age and traffic volume for a variety of highway types.

ACKNOWLEDGMENTS

The writers extend their thanks to Paul Skeels of the General Motors Corporation for his cooperation in running the first statewide survey of pavements in Virginia using the GM skid trailer. Also, the writers acknowledge their liberal borrowing of data from the works of J. H. Dillard, F. P. Nichols, and the late Tilton E. Shelburne who conducted many of the early pavement friction studies in Virginia.

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Appendix

PROCEDURES USED FOR ANALYSES OF TOTAL INSOLUBLES AND CALCITE-DOLOMITE RATIOS

Insoluble Residue Analysis

1. Collect field samples of about 500 grams from each important lithologic unit exposed in a given quarry.
2. Saw a thin slab perpendicular to bedding from each sample; mark and store for thin section use.
3. Crush the remaining sample so that all material is less than ½ in.
4. Split the sample to approximately 200 grams, weigh accurately, and put into a clean, labeled ½-gal jar.
5. Add 400 ml of water and a slight excess of concentrated HCL over the amount needed (approximately 1 ml/gram of rock) to react with the available carbonate. Stir the mixture over a period of days until all reaction ceases.
6. Wash the insolubles free of excess ions by filling the jar with tap water and allowing all of the material to settle (about 48 hours); pour off the clear solution. Repeat procedure 3 times.
7. After the third cycle, wash the insolubles into an evaporation dish, dry at 100 C, and weigh.

Calcite-Dolomite Determinations

Two methods have been used in Virginia to determine the relative amounts of calcite and dolomite in the carbonate fraction.

X-Ray Method—This is essentially the method of Tennant and Berger (12). It involves mixing known amounts of finely powdered calcite and dolomite to form standard samples. These samples are then X-rayed by using a diffractometer with strip chart recorder. The areas under the major peaks of calcite and dolomite are measured, and a standard curve is constructed to show the peak area ratios versus the known calcite-dolomite ratios. The unknown field samples are then powdered and X-rayed. The peak area ratios of the unknowns are determined and compared to the standard curve. This method is accurate to about 5 mole percent.

Chemical Method—This method involves the determination of calcium and magnesium by EDTA (7). All of the magnesium and calcium thus determined are equated to percentages of $MgCO_3$ and $CaCO_3$. A mole percentage of $CaCO_3$ equal to the total mole percentage of $MgCO_3$ is then subtracted from the $CaCO_3$ value and added to the value for $MgCO_3$ to get percentage of dolomite. (This is done by multiplying the percentage of $MgCO_3$ by 100.09/84.33 to get the mole percentage of $CaCO_3$ equal to that of $MgCO_3$. This value is then added to the percentage of $MgCO_3$ to get percentage of dolomite in the rock.) The remaining $CaCO_3$ is considered to be calcite. Such an approach does not allow for the possible existence of a solid solution of Mg in calcite or Ca in dolomite. However, in the Paleozoic carbonates studied in Virginia these effects were small enough to be disregarded.

A Laboratory Method to Determine Polish Susceptibility of Aggregates

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•FROM AN examination of the record of skid tests run in the field, a pattern emerges by which we can associate certain aggregates with slippery pavements and others with pavements that show high skid numbers. At the same time, none of the usual quality tests in our specifications seems to relate directly or even indirectly to these tendencies. Even generalizations based on rock types fail to stand up under close scrutiny.

For obvious reasons, it would be useful to be able to predict in advance that a given aggregate or combination of aggregates can be expected to cause slipperiness. As an extension of this thought, it would also be nice to be able to prescribe corrective treatment for problem aggregates during the mix design stages.

With this in mind, we have attempted to devise a laboratory procedure that will allow us to evaluate the polishing tendencies of aggregates. As a matter of fact, the building of a polishing machine is really no trick at all; however, we are finding that the evaluation of the results is quite another matter.

In the design of our machine, the aim has been to reproduce the action of traffic in the field as faithfully as possible. We, therefore, started by using normal automobile tires as the polishing instrument. Although we have tried several types, we have settled on the use of two 7:50 × 14 ASTM E-17 test trailer tires that have been worn beyond the point of use in the field.

The wheels on which these tires are mounted are at opposite ends of an axle approximately 6 ft long. The system is made to rotate about the midpoint of this axle. It is here that we part company with real-world conditions because this results in continuous turning action with a 3-ft radius. However, it is this tight radius that gives us the polishing action we need. The outer edge of the tire tread actually travels about 14 in. farther than the inner edge on each revolution. This does not sound like much, but it amounts to 6 percent of the total circumference. Our drive speed is such that the wheels go around the 6-ft diameter track at 22½ rpm. This means that in a 24-hour day we produce our 7 miles of drag per tire.

The actual scuff pattern beneath the tire is rather more complex than this because the inner edge is in fact being dragged backward at the same time that the outer edge is being dragged forward. There is also an in-and-out transverse component because any point on the tire is being pulled toward the axis of rotation from the time it comes in contact with the pavement until the hub of the wheel is directly overhead, then that point is pushed outward as rotation continues.

We can, and do, further complicate matters by adjusting the toe-in or toe-out of either tire relative to the axle. We can also alter the speed of rotation and add additional weight to the 300 lb/wheel we normally carry. As you might expect, each of these variables does affect the rate of polish.

In essence, the polishing instrument is a pair of 7:50 × 14 tires chasing each other around a 6-ft diameter circle at about 22½ rpm day and night (Fig. 1).

We have divided the 6-ft diameter track into 16 equal segments, each of which holds one sample. This enables us to conduct replicate tests on a single aggregate and to test more than one material at a time. In comparing data from any given run, we feel

Paper sponsored by Committee on Bituminous Aggregate Bases and presented at the 49th Annual Meeting.

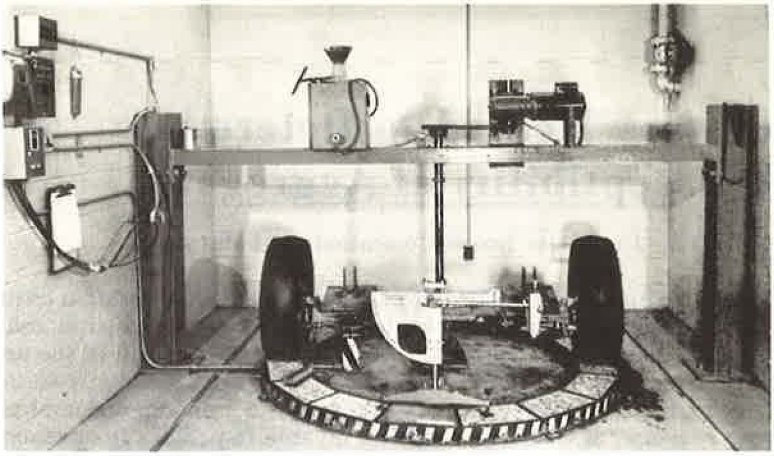

Figure 1.

that we have cancelled out the influence of the several variables in machine adjustment mentioned earlier. We hope to be able to isolate and evaluate these variables at a later date.

Individual samples are trapezoidal in shape, approximately 6 in. wide with a median length of 12 in. (Fig. 1). They are secured in individual metal frames that are bolted into the track. It is convenient to make them 1-in. thick, but provision has been made to increase this up to 3 in.

Although we recognize that sooner or later we will reach a point at which we want to test paving mixtures of various types, we have set this problem aside for the moment, having found by preliminary tests that it can be done.

Currently we are testing the polishing proclivities of aggregates alone. One of the questions we are attempting to resolve is the influence, if any, of the size of the individual particles on the system. For this study, samples of 3 types are being prepared. One type is made up entirely of particles passing a $\frac{3}{4}$-in. sieve and retained on a $\frac{5}{8}$-in. sieve. The second has only material between $\frac{3}{8}$ and $\frac{1}{4}$ in., and the third type is a single slab sawed from a large rock.

In the preparation of individual aggregate specimens, we have devised a method through which the bonding agent holding the aggregate particles is kept well below the plane of the sample surface. We have found that our results are more consistent if we take pains to have one relative flat face of each individual particle uppermost, and arrange it so that these faces are in the same plane.

To accomplish this, we carefully select particles of the size we wish to test and hand place them face down on the flat bottom of a watertight mold as close together as possible. Then, enough water to get a depth of about $\frac{1}{8}$ in. around the aggregate particles is poured in. The mold is then placed in a freezer at 29 F. Once the water has frozen, an epoxy is poured over the frozen surface, and the whole works is returned to the freezer. The temperature is again held at 29 F.

We have found that the epoxy will set up at this temperature in about 2 days. Lower temperatures would probably work but would extend the curing time; higher temperatures melt the ice because of the heat of reaction of the epoxy.

After the epoxy has cured sufficiently to hold itself and the aggregate together, the specimen is removed from the freezer and the ice is washed away. It is then placed face down on a flat plate and allowed to cure for 1 day in an oven at 110 F. This is necessary to maintain the plane surface because the epoxy is a bit on the limber side as it comes from the freezer. The sample is next mounted into the metal frame in a bed of sand-cement mortar. Once this has cured, the sample is ready to be placed in the track.

Incidentally, we are currently experimenting with the use of portland cement and retarding agents in order to simplify this procedure.

Once the specimens are mounted in the track, we expose them to continuous traffic at the rate of 32,500 revolutions, which produce 65,000 coverages per day. Three days a week, usually Monday, Wednesday, and Friday, are set aside for "pit stops" during which we measure the skid resistance of each individual sample.

As implied earlier, anyone can build a polishing machine, but the evaluation is much more difficult. Our measurements of the degree of polish are being made by the use of the British portable tester (BPT).

A template has been devised that enables the operator to set up over the same 2 spots on each sample every time tests are made. The readings from these 2 spots are averaged to give one value for each specimen. This value is combined with similar ones from replicate samples to give a single number representing the degree of polish of the material at the number of wheel coverages completed at any given pit stop.

On 5 different test series so far, we arranged our testing program so as to allow an analysis of variance of the process. Quite similar results were produced in each case. The components examined were between operators, between samples of the same type, between locations on the same sample, and a residual variance.

Two things of interest show up in these studies. First, we are reassured that our data fall in line nicely with the precision statement for the test method as it is given in ASTM. Second, the variance between samples makes up the greatest part of the total to the extent that it is about 4 times the sum of the rest. This finding dictates the use of a number of replicates in order to produce an average result in which we can place some degree of confidence. It also dictates a continuation of our efforts to refine the sample preparation process and the measuring process. The fact that a full test series in any single pit stop takes 2 men about 5 hours also leads us to seek new methods of evaluation.

In spite of this, we are encouraged by the results of our tests thus far. When we plot the average skid number for a material against the corresponding traffic as measured by machine revolutions, a consistent curve emerges. This curve appears to be hyperbolic. So we have set up a program through which the computer gives us the best fitting hyperbola for each set of data points. Correlation coefficients are usually 0.96 or higher. These calculations also produce an intercept at infinity, and we are interpreting this as being the ultimate skid number that would be produced if the polishing process were continued for an infinite number of revolutions. It is the asymptote that the hyperbolic curve approaches with increased traffic. An additional constant produced is a slope from which we may be able to infer a comparative rate of polish.

If it holds up through the rest of our investigation, this certainly gives a convenient quantitative evaluation of ultimate polish potential of an aggregate.

Although we are not ready at this time to release data condemning or praising any given aggregates, I can give a rough idea of the sort of information we are producing. In a recent test series of a Gabbro aggregate known to give quite satisfactory skid results in the field, we ran concurrent tests of 6 samples each of coarse and fine particles along with 4 slab samples.

After 225,000 coverages, all 3 types had dropped from a skid number near 55 to 49. From this point the slab samples began to diverge from the other 2 types until the series was stopped just short of 3 million. During the run, 19 data points of BPT number versus revolutions were made. At the last pit stop, the coarse and fine particle samples each averaged 37, while the slabs averaged 30. The computer showed correlation coefficients of 0.992, 0.988, and 0.996 for the 3 curves representing this series. Calculated values from these best-fit equations gave standard errors of 0.63 to 0.76 skid numbers. The calculated ultimate skid values were 30.5 for the coarse, 27.0 for the fine, and 11.7 for the slabs.

Figure 2 shows the data from the tests of this set of slab samples on a normal linear plot of BPT versus revolutions. In Figure 3, the same data are shown and the hyperbolic function is used for the revolutions scale. The linearizing effect of the transformation of the data can readily be seen.

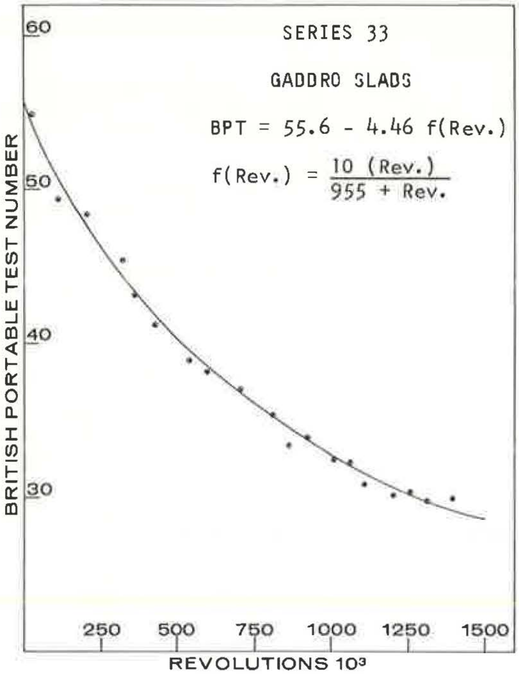

Figure 2.

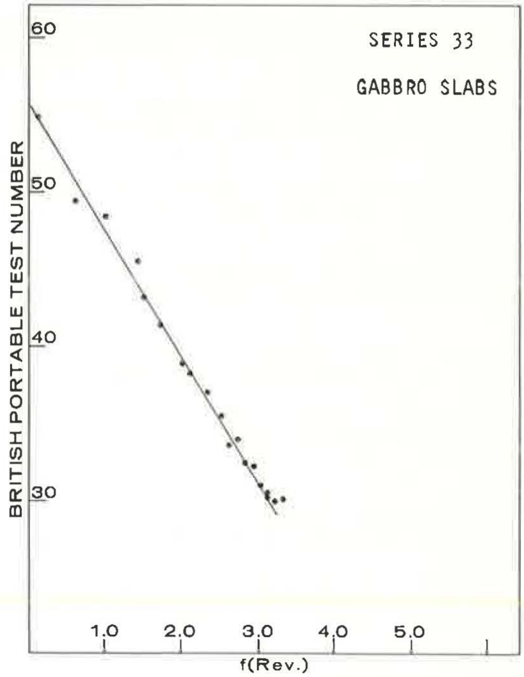

Figure 3.

We are in the process of rebuilding the drive mechanism in order to allow heavier loads to be placed on the tires. At the same time, we have designed and are putting together a device providing a new approach to laboratory skid measurement. By its use we hope to speed up the measuring process and at the same time reproduce the action of skidding in the field more faithfully than we have done with the pendulum type of tester. In the meantime, we intend to continue as we are going and will gladly accept any old, used E-17 test tires anyone wishes to dispose of.

A Review of European Practices for Laboratory Evaluation of Aggregate Polishing Characteristics

R. E. OLSEN, Federal Highway Administration

•IN OCTOBER 1969, I had the privilege of going to several European countries under the sponsorship of the International Road Federation to review research work in the field of skid resistance of highway pavements. I spent a week at the British Road Research Laboratory at Crowthorne, a day at the Central Laboratory for Roads and Bridges in Paris, a day at the Center for Road Research in Brussels, and 2 days at the Technical University in Berlin, West Germany.

I was impressed by the great concern the Europeans have for skid resistance of highway pavements. They have a right to be concerned; the roads are narrow, the traffic is dense, the speed is relatively fast, and the driver is undisciplined by our standards. However, the fact that I did not see an accident during the 2 weeks I was in Europe proves that these drivers, or their opponents, as the case may be, are excellent operators.

Investigations of the skidding problem have been under way in Europe for some time. In 1929 the British developed a skidding test for pavement surfaces using a motorcycle with sidecar. Today they have a truck-mounted, fifth-wheel apparatus with measuring and recording devices that measure the sideways force coefficient of wetted pavements. These units will be allocated to each highway group for continual monitoring of the skid resistance of pavements, and roads or road sections where skidding values fall below minimum values will be promptly resurfaced.

In 1930 test sections were laid to compare material and design variables. The British are currently continuing such investigations for a better nonskid type of surfacing from both the design and construction points of view, that is, size of stone, rate of application, and special binder. They are also investigating special types of manufactured aggregate. A specific source of calcined bauxite has an exceptionally high resistance to polishing under traffic. However, at this stage it is very expensive—about $75/ton of usable material. Several test sections were laid and, as measured by the British pendulum tester and sideways force coefficient apparatus, the calcined bauxite proved to be better than the best of natural roadstone. This material has a polished stone value of about 75.

It is the hope of the research to find a suitable manufactured aggregate made up of a matrix containing granules of another material that is hard and angular. The ideal matrix should be cheap, be easily mixed with the abrasive granules, have satisfactory strength, and be one that will wear away under traffic at an optimum rate, thus exposing new grit particles. The researchers are of the opinion that the microtexture of the aggregate particles is important but that the road surface should be rough textured.

Road surface texture measurements are being mady by using stereophotographic techniques with computer analysis. A "profile ratio" value is determined, which is the ratio of the length of the profile to the length of the base line. This procedure, however, does not indicate the state of polish of the surface aggregate. Other techniques of assessing the surface roughness are now under investigation. These include

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angularity of the peaks of the exposed aggregate, distribution of projection, and tire penetration. No satisfactory method has yet been developed to determine tire penetration.

The Road Research Laboratory and the Cement and Concrete Association of the United Kingdom have developed an accelerated wear machine for studying portland cement paving mixture. It is, in reality, an enlarged version of the stone-polishing apparatus. The wheel containing the test specimens is about 15 in. in radius, holds 16 specimens that measure about $6^3/_8$ by $3^1/_2$ in. and is driven at 150 rpm. Two smooth rubber tires are set at 2 deg out of line with the test wheel. The specimens are abraded dry with a fine flint sand for 50 hours and then wet abraded with a fine emory powder for 5 hours. The specimens are tested for skid resistance with the British pendulum tester.

Studies have been made with this test procedure on the influence of types of texturing devices such as wire brooms, rubber rakes, and rollers; the application of polish resistant chippings; the assessment of the relative merits of various fine and coarse aggregate; and the influence of mixture design.

Some general conclusions from these studies indicate the value of a deep and rough texture in the surface of portland cement concrete for high speed traffic; however, such a texture wears very rapidly in heavy traffic. Increasing the cement content and strength of concrete mixtures resulted in a higher degree of polish and, therefore, poorer skid resistance; however, such mixtures retain their rough texture longer. It has been shown that the most important characteristic of the aggregate is the hardness of the fines and that the hardness of the coarse aggregate is of secondary importance.

The French are currently working on surface texture measurements. Their technique is to make a rubber casting of the surface and from this mold a replicate of the surface in epoxy resin. These molds are surprisingly very true to the original surface. The castings are cut so as to show the profile of the pavement, and this is magnified on a screen. The profile ratio is measured and analyzed by computer.

The French are also studying road surface drainage. They are in the laboratory stage of testing small pavement samples for the amount of water retained at variable rates of water application. The relation among slope of the pavement, surface roughness, quantity of water retained on the surface in equilibrium, and quantity of water trapped in the pavement and retained by absorption are being investigated. They are also determining the mean depth of water films on pavement surfaces with nuclear devices.

In the field of portland cement pavements, the French advocate mechanical brushing in the transverse direction to remove surface laitance and choosing a mortar that does not polish. This is accomplished by using a very hard and angular sand so that differential wear of the mortar will leave the sand particles exposed from the cement paste.

The French use the British stone polishing test. They find that it does correlate well with pavement condition and use it for aggregate evaluation. However, they recognize the shortcomings of the test; that is, the shape of stone can affect results and the stone layout or pattern is not realistic. They have found good correlation between the polishing characteristics of aggregate and the Vickers hardness test.

In Belgium studies are concerned with the techniques of texturing portland cement concrete. The Belgians too have developed means to incorporate transverse grooves in fresh concrete and have found them to be effective. One study that has been going on for the past 6 years has shown that the coefficient of friction has remained quite constant at 0.65 to 0.75 at 30 mph. The change in coefficient when testing at slow and fast speeds is less than 0.2. They have found that the decrease in the coefficient of friction with increasing speed is linear with the depth of the groove and is independent of the degree of polishing of the pavement. However, at slow speed the coefficient is independent of the texture and is dependent on the degree of polishing.

In Germany there is interest in profile measurements made with a rubber-coated stylus. The stylus is slowly moved across the pavement surface and reacts to the size and distribution of surface asperities. The force necessary to move the stylus is also measured, and this can be related to the coefficient of friction of the pavement surface. This method of measurement is believed to take into account the microroughness of the surface of the pavement. It is believed that from these data good predictions of the coefficient of friction of the pavement surface at 50 mph can be made. At the Technical University, studies are under way to investigate the forces and mechanics involved in the phenomenon of hydroplaning. Accident data are being collected to spot locations where skidding accidents occur and to investigate these pavements for characteristics that involve skidding. The Germans believe that accident statistics are the most important parameter when judging the need for resurfacing to improve the resistance to skidding.

Effect of Aggregate Mineralogy on Polishing Rate and Skid Resistance in Pennsylvania

WADE L. GRAMLING, Pennsylvania Department of Transportation

•THE PENNSYLVANIA Department of Transportation began its research in skid resistance in 1960 with a series of pavement friction tests in a limited area of the state. This pilot study revealed that of 19 sites selected for suspected slipperiness 11 were classified as slippery; the pavements at these 11 sites exhibited polished carbonate aggregate. A continuing program of measuring skid resistance on Pennsylvania highways has been conducted in annual skid surveys following the recommendations of the study report, and formal research projects dealing with slipperiness and polishing have been under way.

A review of the data collected from this work by early 1968 indicated that there was no satisfactory construction or material specification to ensure that a bituminous pavement surface would have adequate skid resistance throughout its life.

Figure 1 shows the results obtained from the annual skid surveys taken from 1962 to 1967. The 687 tests shown were subjectively chosen for suspected slipperiness and would not necessarily be representative of the skid-resistance values of Pennsylvania highways. The skid resistance of normal bituminous concrete pavement surfaces in Pennsylvania is generally related to the type of coarse aggregate used. Of the pavements tested, the average skid resistance of the surface courses constructed with gravel and sandstone are considerably higher than those constructed with carbonates. Where slag and other types of aggregates have been used, intermediate skid measurements have been obtained.

Although average skid values are shown for each of the general types of aggregate, a wide range of values was found within each type. The controlling aggregate parameters have not been clearly defined within each type to permit specifications to be written that would accept all those aggregates giving adequate skid resistance and omit those polishing excessively.

In order to evaluate these parameters in carbonate aggregates, a series of test sections was planned and constructed.

TEST STRIP PLANNING

As the skid research data accumulated, it became apparent that the coarse aggregate in a bituminous mix was the most influential in contributing to skid resistance. Other factors such as surface type of gradation have lesser significance. Laboratory polishing work generally agreed with the field observations, showing that relatively pure carbonate aggregates polish uniformly and become slippery but, as the amount of insoluble sand-sized material within the aggregate particles increased, the skid-resistance properties improve.

A review of the approximately 340 sources producing aggregates showed a wide variety of properties. It is imperative from an economic standpoint that all sources that could supply aggregates providing an adequate skid-resistance level throughout a normal pavement surface life be permitted in any specification.

Paper sponsored by Committee on Bituminous Aggregate Bases and presented at the 49th Annual Meeting.

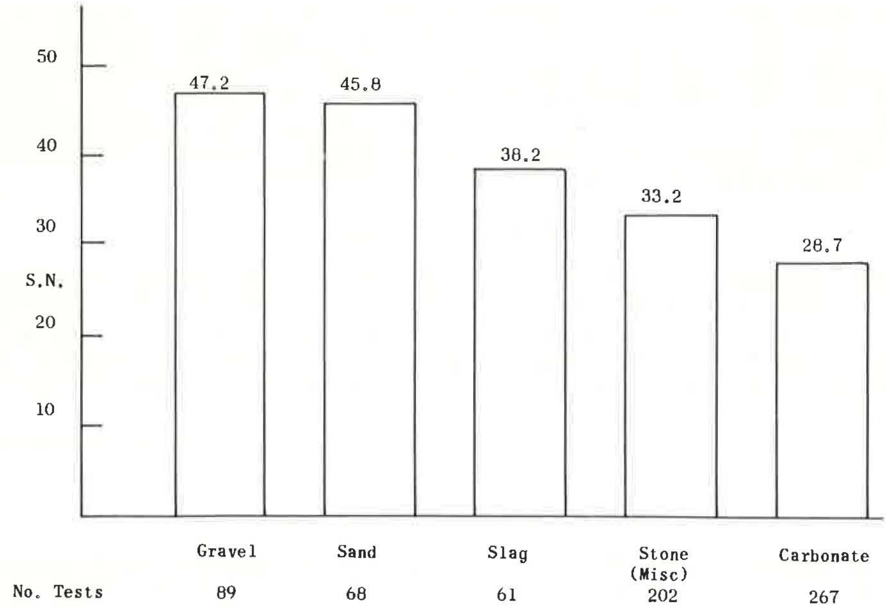

Figure 1. 1962-1967 skid surveys.

Insoluble residue tests were run on a cross section of carbonate aggregates produced from rock from various geologic areas in Pennsylvania. This permitted a selection of 20 carbonate aggregates, including limestones and dolomites with insoluble residues ranging from less than 1 percent to more than 35 percent as shown in Figure 2.

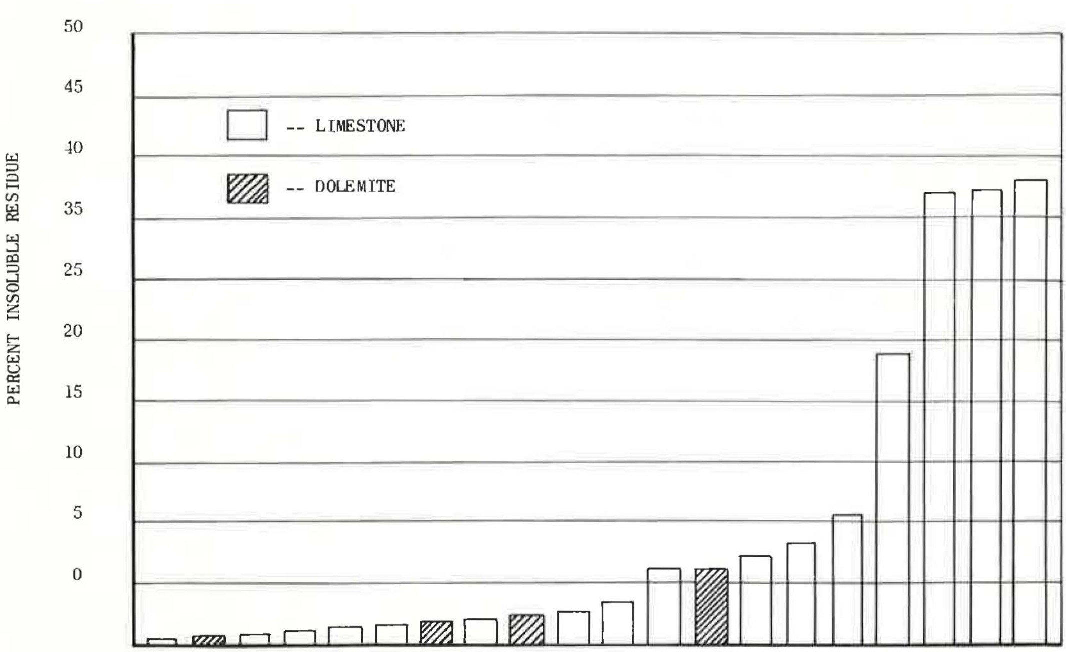

Figure 2. Percentage of insoluble residue of +200 sieve size on test strips 1 through 11.

Other representative types of aggregates produced in the general location of the 20 carbonate aggregates selected were chosen for incorporation into the test strips to permit determination of the skid levels they might produce under similar conditions. Each test strip then included one or more of the carbonates, gravel, slag, and other rock types as available. A number of blends of carbonates with other aggregates were also included to determine the effect of blending on skid resistance. A minimum length of 1,000 ft was set for each section to allow for skid testing.

TEST STRIP SITES

The test strip road locations were selected from projects planned as part of the normal construction program in order to expedite construction. The sites were selected with sufficient length to accommodate all planned sections in one lane and to have a reasonably high traffic count uniformly throughout the length. A minimum of side traffic turning movements was wanted, and excessive grades and curves were avoided so that uniform polishing would be obtained.

The criteria resulted in 11 test strip locations to accommodate all the aggregates selected.

TEST STRIP CONSTRUCTION

The test pavements were constructed during 1968 and 1969 by using Pennsylvania ID-2A hot-mix surfaces that have a maximum ½-in. size of aggregate. All of the mixes for each of 10 of the test strips was supplied from a single plant in the area. Regular production procedures were used, with only the substitution of the coarse aggregate that was supplied from the selected source. The fine aggregates were those normally used by that plant. The eleventh test strip had material supplied from a number of plants in addition to the one plant supplying mixes with all of the selected aggregates.

TEST STRIP AGGREGATES

Samples of all of the aggregates used in the wearing courses of the test strips have been examined petrographically and identified for stratigraphic unit. Acid insoluble tests have also been run.

Information for the test strip aggregates is given in Table 1. Over 43 lane-miles of pavement were required to accommodate the 156 aggregate sections containing 64 sources and blends.

Figure 3 shows a typical test strip plan. It shows the random arrangement of test sections within a lane and each test section occurring in both lanes. The same section is repeated in the same lane at several sites to provide replicates, and several test aggregates are included in 2 sites to evaluate the effect of geographic location and other variables.

EVALUATION PROGRAM

Traffic counts have been obtained on the 11 test strips, and a periodic program of skid testing on each of the test sections is under way. These data will permit the development of a skid history curve for each aggregate by plotting accumulated traffic passes versus skid number. Table 2 gives the type of data being collected. The numbers shown are purely for illustrative purposes and have not been corrected for surface temperatures or evaluated for weather conditions.

Both core and slab samples have been taken from each test section and are being investigated in laboratory work to evaluate polishing and test techniques. This phase is directed at devising quick laboratory methods that will predict field performance of aggregates.

TABLE 1

TEST STRIP MATERIALS

Type	Source	Section	Length (mile)
Carbonates	20	59	25.43
Slags	8	20	3.73
Gravels	9	18	3.83
Others	12	30	5.62
Blends	15	29	5.27
Total	64	156	43.88

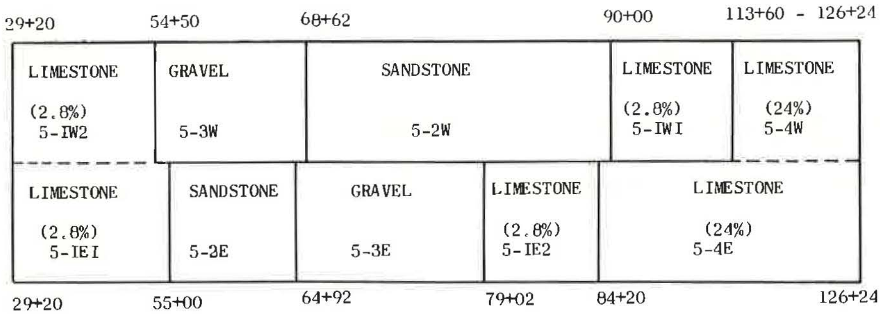

Figure 3. Plan of test strip 5.

The slab samples are placed under a loaded test wheel that spins in place at a constant speed and load. A planned cycle of water and sand is introduced between the tire and the specimen, and the tractive force on the sample is recorded over several hours of running. A steady state of force is reached at the end of the test.

TABLE 2

DATA COLLECTED FOR TEST STRIP 5

Item	Test Date	
	10-23-68	9-10-69
Traffic passes	180,000	1,800,000
Aggregate, average skid reading		
Limestone, 2.8 percent	48	42
Silt-sandstone	49	52
Gravel	48	51
Limestone, 24 percent	48	42

The core samples are being evaluated in an apparatus developed in earlier polishing studies at Pennsylvania State University. This technique uses a reciprocating rubber shoe sliding over the core and measures the power required for polishing. Tests are also made of the final friction values.

Samples of aggregates used in the test strips are undergoing tests in a drum apparatus. The aggregates are glued to metal sections that are mounted as segments on a revolving drum that runs against a polishing tire. Friction values are also obtained periodically and at the conclusion of polishing.

SUMMARY

The aggregates included in the test strip program represent a cross section of the rock types commonly used in the wearing surfaces of pavements in Pennsylvania. The aggregates have been identified petrographically, and their properties have been determined. Continued exposure to traffic and period skid testing should provide an evaluation of the skid-resistance potential of many of the rock formations in Pennsylvania. Suitable aggregate specifications can then be prepared that provide adequate skid resistance throughout the life of the pavement.

The supplemental laboratory testing of the same aggregates is directed at developing a method to allow prediction of field performance of any aggregates proposed for use.

Michigan's Experience With Different Materials and Designs for Skid Resistance of Bituminous Pavements

PAUL J. SERAFIN, Michigan Department of State Highways

•WITH THE rapidly rising number of vehicles on the road, it is not surprising that there is a corresponding rise in the number of vehicular accidents. Factors contributing to this include human behavior, vehicle mechanical failure, environmental conditions, road layout, and pavement surface. To tackle all these problems is a task for people of diversified interests. This report intends to isolate one of these factors, pavement surface, and to report on the work performed during the past 4 years by the Michigan Department of State Highways in improving skid resistance of pavements.

Accumulations of deposits on the road surface, such as rubber particles from tires, mud film, and oil and grease drippings from auto crankcases, are among the factors that contribute to slippery pavements, especially at the beginning of a rain after a long, dry spell. Excess surface bitumen and polishing of the coarse aggregate particles also contribute measurably to slippery pavement conditions.

An example of a slippery pavement caused by polished coarse aggregate is shown in Figure 1. The reflection of light from the pavement surface is a good indication that it may be slippery. A close-up of a sawed pavement core taken from this surface is shown in Figure 2. The coarse aggregate particles have become oriented in such a way that their flat faces are exposed to the surface and subject to subsequent polishing by traffic. When this same specimen is turned at an angle, as shown in Figure 3, one can see the oily bitumen absorbed into the surface of the coarse aggregate particles. This adds to the problem under wet conditions.

Although all coarse aggregate particles will eventually polish and exhibit these characteristics, experience indicates that some carbonate aggregates such as limestone, particularly the softer type, will polish at a faster rate than the harder or noncarbonate aggregates. On the other hand, experience indicates that fine aggregate particles produce a sandpaper surface texture that offers skid resistance. As the fine particles are gradually lost through wear, the surface rejuvenates itself and the sandpaper texture continues.

Although a sandpaper surface texture offers a skid-resistant surface, the effect of this sandpaper texture becomes minimized as vehicle speeds increase, particularly when the pavement is wet. Some engineers explain this as being due to hydroplaning of the tire on a water film. To nullify this, one might consider constructing a surface having sharp aggregate particles that protrude through the water film to make contact with the tire at higher speeds.

Most accidents occur in heavy traffic as vehicles approach intersections or other critical areas that may cause drivers to brake. Normal maximum speeds in these areas are below 40 mph. Therefore, in the interest of confining ourselves to a specific type of condition, let me further isolate this problem to surfaces that will be subjected to speeds under 40 mph.

Correcting these critical conditions usually involves resurfacing, commonly performed by placing a bituminous mat on the old surface. So let us further narrow the scope of this study to fine-textured, bituminous, skid-resistant surfaces.

Paper sponsored by the Committee on Bituminous Aggregate Bases and presented at the 49th Annual Meeting.

Figure 1. Slippery pavement surface of polished limestone.

Figure 2. Pavement cross section showing oriented flat particles.

Figure 3. Pavement core showing aggregate particles in oily polished surface.

A statewide search was conducted to obtain different types of hard aggregates that could be crushed and used to produce fine-textured, bituminous surfaces. Also, preliminary laboratory studies were made to consider maximum particle size, gradations, bitumen content, mineral filler content, air void, and stability. To further supplement this investigation, a contract was entered into with the American Oil Company to allow preliminary skid studies to be conducted on its circular test track over short sections using different mixes.

Based on information obtained from all of these sources, a number of different types of materials and designs were used to produce skid-resistant surfaces at selected critical areas in the southern part of Michigan. The following are some of the materials used in these installations. A brief description is given of each.

SANDSTONE

This aggregate was a sandstone from the Grindstone City area of Michigan's "thumb" region; it is geologically identified as the lower Marshall sandstone formation, Pointe Aux Barques members. Large broken pieces of old, seasoned, discarded grindstones, which were processed years ago, were used as raw material for this project. Pieces containing excessive amounts of the finer grained brownish layers were generally not used. Freshly quarried sandstone from this source is reported to be somewhat "friable," and may require seasoning in the open air to harden the cementing material that holds the sandstone grains together.

The material was put through a crusher, and that portion still exceeding 3/8 in. was passed through the crusher a second time. These 2 crushings were combined and resulted in the following average gradation:

Sieve	Percent Passing
3/8 in.	100
No. 4	64
No. 8	44
No. 30	35
No. 100	24
No. 200	8

Physical characteristics of the material tested in the central laboratory were as follows:

Soundness, percentage of loss	57
Los Angeles A abrasion, percentage of wear	59
Specific gravity, dry basis	2.2
Absorption, percent	7.2

Figure 4. Crushed sandstone surface 4 years after construction.

Figure 5. Close-up of sandstone surface 4 years after construction.

Because the crusher-run material appeared to contain sufficient fines, no additional sand was added to the initial mix design. Several bitumen contents, varying from 8 to 10 percent, were tried, but the resulting mixture was tough to handle and produced a nonuniform, open-textured mat. After some experimentation, the mix design was changed by adding 1 part fine sand to 5 parts crusher-run sandstone and 9.5 percent 60 to 70 asphalt. Commercial mineral filler was not added to this mixture. This mixture produced a uniform-textured, skid-resistant surface and was used on 3 intersections in the Bay City area. Normally, a gradation of this type would use about 6.5 percent asphalt; however, because of the high absorption of this material (7.2 percent), it was necessary to raise the bitumen to about 9.5 percent in order to produce a serviceable pavement. Figure 4 shows a typical intersection after 4 years of service where this sandstone mixture was used, and Figure 5 shows a close-up of the sandpaper surface texture. After several months of wear, the larger sandstone particles began to show a dishing effect as shown in Figure 6. This has not detracted from the excellent service performance of this surface after 4 years of service under moderately heavy traffic; rather, it is felt that skid resistance characteristics have improved.

QUARTZITE

The aggregate used is identified as Ajibik quartzite and was obtained from the Upper Peninsula. The crusher-run material contained about 17 percent passing the No. 200 mesh sieve, which was considered somewhat high for a proper mix design. Consideration was given to washing a portion of the crushed material and blending the washed and unwashed stockpiles in proportions that would produce about 7.5 to 10 percent passing the No. 200 sieve. Inadvertently, the producer washed all the crushed material produced, resulting in the following gradation:

Sieve	Percent Passing
No. 8	100
No. 16	72
No. 30	43
No. 50	25
No. 100	11
No. 200	4.5

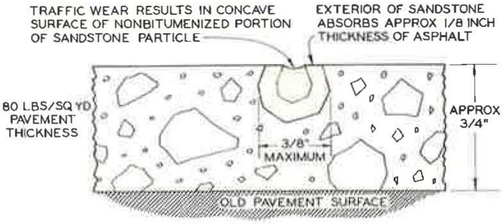

Figure 6. Result of traffic wear on larger particles of crushed sandstone.

Based on laboratory-prepared Marshall specimens and the appearance of a trial pavement surface, the mix design for the major portion of the project was set at 6.5

Figure 7. Crushed quartzite surface 3 years after construction.

Figure 8. Close-up of quartzite surface 4 years after construction.

percent 60 to 70 asphalt; 4.0 percent fly ash was added as mineral filler to make up for the fines lost through washing, and the remainder was quartzite crushings. No other aggregate was added for this mix design. Figure 7 shows a typical surface after 3 years of service in the Flint area. Some loss was experienced where traffic entered from a side gravel road causing considerable localized abrasion; however, the rest of the surface is in good condition. Figure 8 shows a close-up of this excellent skid-resistant surface.

Stripping tests performed on quartzite have usually indicated poor adhesion of the asphalt to the quartzite surface in the presence of water. The amount of stripping may vary, depending on the sources of the materials used. For this reason, on one additional project a heat-resistant, anti-stripping material was added to the asphalt cement before incorporation in the mixture. An intersection north of Flint was selected for this application to determine by actual service whether any differences show up after service between the quartzite mixtures containing asphalt with and without the additive. One percent of heat-resistant, anti-stripping additive was added to the 60 to 70 asphalt cement. A stripping test was performed in the laboratory on a mixture prepared with this quartzite (P8-R16) and 4.5 percent asphalt cement containing the additive. After curing, the mixture was subjected to a boiling test. Eighty percent of the asphalt containing the additive was retained on the quartzite, as compared to only 20 percent of the asphalt without the additive on a similar mixture. After 4 years, the surface containing the additive appears to show somewhat less wear than the surfaces constructed with untreated asphalt. However, both areas exhibit satisfactory durability and excellent skid resistance.

CRUSHED BEACH PEBBLES

The aggregates used were crushed pebbles obtained from beach deposits on the Lake Superior shore at the east end of the Upper Peninsula. After many years of grinding wave action on the aggregate particles, only the hard pebbles remain. The deposit is quite extensive along the lakeshore in this vicinity and is used as a source of pebbles for polishing, for decorative panels, and for other applications.

It was desired to crush the pebbles, ½ in. or larger, to 100 percent passing the No. 8 sieve; however, difficulties were experienced with the small crusher used for this purpose. Therefore, it was decided to accept the material as produced with the following average gradation:

Sieve	Percent Passing	Sieve	Percent Passing
⅜ in.	100	No. 30	30
No. 4	97	No. 50	19
No. 8	70	No. 100	11
No. 16	47	No. 200	5.5

Figure 9. Crushed beach pebbles 3 years after construction.

Figure 10. Close-up of beach-pebble surface 4 years after construction.

The mix design used was 5.7 percent 60 to 70 asphalt and 2.0 percent fly ash mineral filler; the remainder was crusher-run beach pebbles. No other aggregate was added to this mixture. Figure 9 shows a typical surface 3 years after construction at an intersection south of Flint. Figure 10 shows a close-up of the surface after 4 years of service. This surface is in fair condition; however, because of relocation of the roads in this area, part of this project was resurfaced last year.

TRAPROCK

The aggregate used was predominantly of basic igneous origin and was obtained from Michigan's Upper Peninsula. Because of the high amount of stone dust, the crushed material was washed, resulting in the following gradation:

Sieve	Percent Passing
No. 8	100
No. 16	76
No. 30	44
No. 50	22
No. 100	10
No. 200	6.6

The mix design called for 6.1 percent 60 to 70 asphalt plus 3.5 percent added fly ash mineral filler; the remainder was crushed traprock. No other aggregate was added to this mixture, which was used at an intersection south of Flint. Figure 11 shows the surface after 3 years of service. Considerable areas of the surface were worn completely through to the old pavement. No detour provisions were made, and heavy traffic was permitted to traverse the uncured, warm mat, resulting in premature wear that was evident within a few days after construction. I believe this mix would have proved more durable if traffic had not been permitted on the warm surface during construction.

Figure 11. Crushed traprock 3 years after construction.

SYNTHETIC BITUMEN AND NATURAL SAND

Previous experience indicated that, when certain synthetic bitumens were mixed with

Figure 12. Synthetic bitumen and sand 1 year after construction.

Figure 13. Sand-emulsion surface 4 years after construction.

sand, they exhibited promising skid resistance because of what appeared to be a noticeable rejuvenation of the sandpaper texture. This mixture was placed at a rate of 80 lb/sq yd on a section of I-96 about 50 miles west of Detroit. During the start of the first winter, sections of the pavement peeled off as shown in Figure 12. The rest of this surface was later bladed off. It was determined that considerable hardening of the bitumen occurred within a few months resulting in penetrations approaching zero. It is believed this contributed to the loss of adhesion between the thin bituminous mat and the underlying concrete pavement.

HOT ASPHALT EMULSION AND SAND

In recent years the asphalt emulsion industry has promoted the use of hot emulsion sand mixtures for skid-proofing purposes, and other states have reported satisfactory service with these mixtures. A mixing grade emulsion was used for this purpose; the producer claimed that the bitumen residue has certain thixotropic characteristics, thus permitting higher bitumen contents to be used in the mixture without danger of flushing. A fine aggregate passing the No. 8 sieve was mixed with about 10.8 percent emulsion resulting in about 7.5 percent residue. The temperatue of the mixture delivered to the street ranged between 245 and 270 F. Figure 13 shows a typical pavement section after 4 years of service, and Figure 14 shows a close-up of the fine sandpaper texture. This road is in good condition and offers a skid-resistant surface.

SAND ASPHALT

Although most of the previous materials and designs exhibited excellent skid-resistant properties, they were costly to construct primarily because of special handling of materials that are not normally used for paving purposes. On the other hand, it has been determined that ordinary sand-asphalt mixtures produce satisfactory although not high skid-resistant surfaces. Because sand-asphalt mixtures are less costly, it was decided to use them for much of the remainder of the skid-proofing program during the next several years.

At the beginning of the first winter season, some sand-asphalt surfaces showed delamination in the wheel tracks; that is, about ⅛-in. layers would separate from the rest of the mat. Figure 15 shows a typical example of such a road, and Figure 16 shows a detail of this phenomenon.

Although some theories have been proposed as to the cause of this, the writer has reservations about these explanations. Of more importance is the fact that solutions have been found to reduce this type of failure. After several years of observation, it was noted that bituminous mixtures with sands that are predominantly a one-sized "belly" type often exhibit this delamination, while uniformly graded sands very seldom show any delamination.

Figure 14. Close-up of sand-emulsion surface 4 years after construction.

Figure 15. Delamination of sand-asphalt surface 1 year after construction.

STONE-FILLED SAND ASPHALT

Another solution to the delamination problem is to tie the ⅛-in. top layer to the lower main portion of the mat by introducing coarse aggregate (⅜ in. top size) into the sand-asphalt mix. Figure 17 shows this technique of tying the upper and lower layers together. Satisfactory performance has been observed when this technique is used with the coarse aggregate content ranging from 10 to 25 percent. Figure 18 shows a pavement in Detroit with a stone-filled sand-asphalt mixture after 4 years of service. Figure 19 shows a close-up of the stone particles sticking up above the sand-asphalt mortar. Because there are very few exposed stone particles, a softer type of aggregate such as limestone may be used without concern regarding the polishing effect.

ASBESTOS FIBER AND SAND ASPHALT

Experience indicates that, within certain limits, leaner mixes exhibit better skid resistance than bituminous pavements designed at their optimum bitumen and mineral filler content. However, these leaner mixtures may show faster wear than corresponding surfaces having optimum designs. In an attempt to improve the durability of these mixtures, short fiber asbestos was added as filler; because of higher absorption, it allows more asphalt to be held in the mix. Three intersections were constructed with this material, all on Telegraph Road in the Detroit area. A typical design used is as follows:

Material	Percent	Material	Percent
Sand	87.5	Fly ash	2.0
Asbestos	2.0	Asphalt	8.5

Figure 20 shows a typical area in Detroit after 4 years of service. It is generally in good condition except for the fillet areas that were manually placed and cooled before

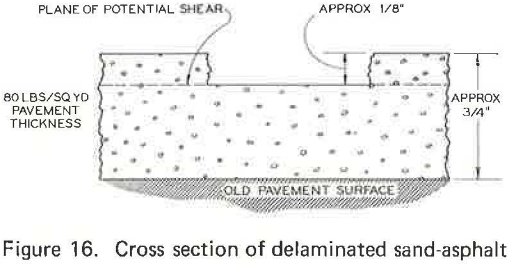

Figure 16. Cross section of delaminated sand-asphalt surface.

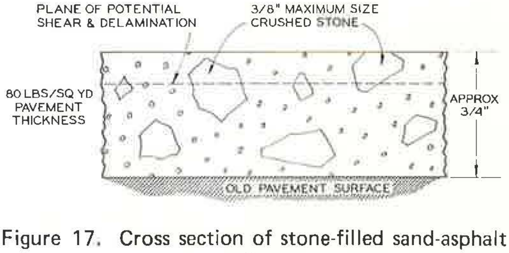

Figure 17. Cross section of stone-filled sand-asphalt surface.

Figure 18. Stone-filled sand-asphalt surface 4 years after construction.

Figure 19. Close-up showing stones sticking out of stone-filled sand asphalt.

they were properly compacted, resulting in some raveling. Figure 21 shows a close-up of the sand-asbestos-asphalt surface texture that exhibits satisfactory skid resistance.

SINOPAL AND SAND ASPHALT

During the period of placing the other experimental skid-resistant surfaces, a synthetic aggregate, which is a product of fluxed silica and dolomite, was shipped in from Europe. This is a white aggregate having a hardness of about 6 on the Mohs' scale of hardness. One part of the synthetic aggregate was mixed with 2 parts of natural sand, to which was added 2.5 percent limestone filler and 6.5 percent 60 to 70 asphalt. This was laid ½ to ¾-in. in thickness in the Benton Harbor area.

Figure 22 shows this surface after 4 years of service. The lighter colored areas shown in the right 2 lanes have the Sinopal added, while the darker areas in the left 2 lanes are mixtures of sand asphalt without Sinopal. Figure 23 shows a close-up of the Sinopal surface; the white particles of Sinopal can be seen exposed at the surface. This surface, in general, is in good condition and exhibits good skid resistance.

CONSTRUCTION NOTES

Motorists are sometimes warned of freshly laid bituminous surfaces by a sign indicating "slippery when wet." Figure 24 shows a freshly laid bituminous mat constructed with MC-3000 liquid asphalt. The left foreground surface was laid the day before, while the rest of the surface, which appears spotted, had just been completed when it was rained on. After 1 day of curing and weathering, water wets the pavement. However, as shown by the close-up in Figure 25, water globules have formed on the mat of the fresh surface. This nonwetting phenomenon of a fresh bituminous surface may contribute to slippery pavements when wet.

Figure 20. Raveled fillets of sand-asbestos surface 4 years after construction.

Psychologically, when a driver observes a smooth wet newly constructed bituminous surface, he may be inclined to apply brakes to slow down. Because of the nonwetting phenomenon and because the sandpaper texture has not yet developed, the driver may lose control of his vehicle on the slippery pavement. Therefore, it was felt that something had to be done to improve the initial skid resistance of pavements that ultimately should offer good skid resistance.

After considerable experimentation, a procedure was developed to produce an

Figure 21. Close-up of sand-asbestos surface 4 years after construction.

Figure 22. Sinopal-sand surface 4 years after construction.

Figure 23. Close-up of Sinopal-sand surface 4 years after construction.

Figure 24. Freshly laid bituminous surface after rain.

Figure 25. Close-up of fresh, oily, nonwetting bituminous surface after rain.

Figure 26. Spreading precoated sand on unrolled bituminous mat.

Figure 27. Spreading precoated sand without deflector (most of sand falls on shoulder).

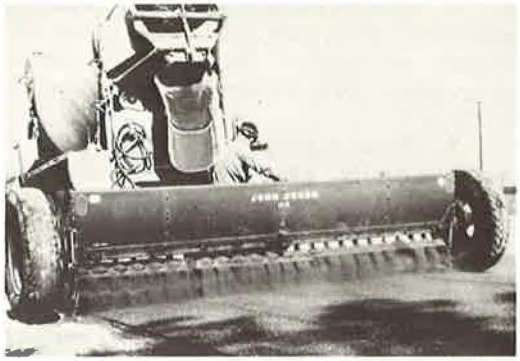

Figure 28. Seeder used for spreading precoated sand leaves ridges.

Figure 29. Rolling pavement after sand application.

Figure 30. Precoated sand on mat of partly rolled surface.

initial skid-resistant bituminous surface by applying hot sand, precoated with about 1 percent asphalt cement, on the unrolled mat behind the paver. This is then rolled into the surface and reduces the initial slickness. About 2 to 5 lb/sq yd of this precoated sand is sufficient; more may produce a ball-bearing effect, which can also be dangerous and may mar the pavement surface when bunched-up sand is rolled into the mat.

Figures 26 through 30 show some typical equipment used in this technique. The spreader shown in Figure 27 is not equipped with a deflector, thus allowing about $^2/_3$ of the precoated sand to be wasted on the shoulder. Using a seeder, as shown in Figure 28, sometimes leaves ridges that subsequently mark the pavement.

CONCLUSION

All the experimental surfaces start out with an initial average skid-resistance coefficient of friction of more than 0.45 (Fig. 31). The sandstone surface had an initial coefficient of 0.74, the quartzite surface of 0.71, the crushed beach pebbles of 0.63, and so on. There is a dip in all the curves after 1 year of service followed by an increase and leveling off of the coefficients. After 4 years of service, the sandstone and quartzite surfaces with coefficients of 0.57 and 0.58 are higher than the rest of the materials used in this study.

Considering that a satisfactory coefficient of friction should be more than 0.35 or 0.40, it is indicated that excellent skid resistance can be obtained by using special hard aggregates in the bituminous surfaces. However, the uniformly graded sand-asphalt mixtures offer a satisfactory skid resistance and have proved through service to be durable. Therefore, currently many of the critical areas on Michigan trunk lines are being resurfaced with the more economical sand-asphalt mixtures.

After observing these bituminous skid-resistant surfaces for 4 years, we feel that the following 10 items are worthy of consideration.

1. Soft coarse aggregates such as the carbonate types may polish faster than the harder aggregates. However, all coarse aggregates will eventually polish under traffic.

2. Fine aggregates (passing No. 8 seive) usually wear away exposing new ones before they become polished; therefore, they offer skid resistance at lower speeds (below 40 mph).

3. Uniformly graded fine aggregates resist wear better than nonuniformly graded ones and are not prone to surface delamination.

4. Angular fine-aggregate particles offer slightly better skid resistance than rounded sands.

5. Natural sand, well-graded and containing predominantly hard particles (5.5+ on Mohs' scale), offers satisfactory skid coefficient (0.40 to 0.50) and may be considered over the special aggregates for economic reasons.

6. Fresh bituminous surfaces are oily and nonwetting and are initially slippery when wet. This may be corrected by the application of sand, precoated with 1 percent asphalt, to a mat before rolling.

7. Mats of 80 lb/sq yd appear to be the desired application rate for skid-proofing purposes. A lesser thickness may wear prematurely and may peal off; a greater thickness may rut or shove.

8. Addition of mineral filler and asphalt should be kept slightly below optimum to obtain satisfactory skid resistance.

9. Thin mats are best applied at 60+ F air temperatures to obtain satisfactory compaction before chilling.

10. Existing surfaces must be thoroughly cleaned and have a light application of tack coat before thin bituminous skid-resistant mats are placed.

	0	1/2	1	2	3	4
	1965	1966 SPRING	1966 FALL	1967	1968	1969
QUARTZITE	71	52	50	57	54	58
SANDSTONE	74	54	48	53	57	57
SAND-EMULSION	59	48	46	54	54	56
CRUSHED BEACH PEBBLES	63	47	46	44	47	52
SYNOPAL-SAND	47	41	39	54	47	49
STONE-FILLED SAND	52	41	37	45	45	48
ASBESTOS-SAND	62	39	36	43	44	46
SAND-ASPHALT	47	41	41	52	47	50

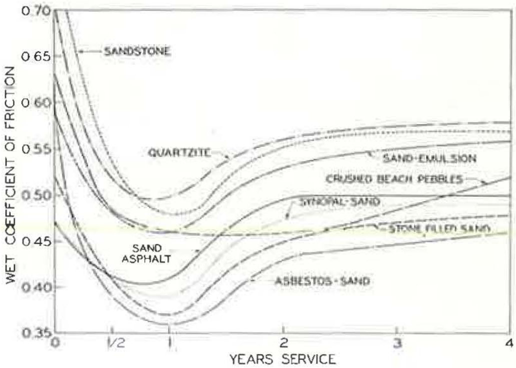

Figure 31. Pavement skid-resistance coefficients of different materials versus years of service.

Macrotexture Measurements and Related Skid Resistance at Speeds From 20 to 60 Miles per Hour

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The role of macrotexture in imparting friction capabilities to pavement surfaces is of major concern to researchers. Macrotexture is but one of the many variables affecting the interaction at the tire-pavement interface; however, at present its relative importance is questioned. Friction tests obtained at 20, 40, and 60 mph with the Texas Highway Department research skid trailer and macrotexture tests utilizing 4 different methods were conducted on 41 pavement surfaces. These surfaces exhibited widely different friction levels, friction-speed gradients, drainage capabilities, mineralogical properties, and texture classifications. Macrotexture values obtained with the 4 methods are compared. The effects of macrotexture types and magnitudes on friction numbers and friction-speed gradients are analyzed. Statistical analyses and typical plots are given. Brief descriptions of several macrotexture measurement methods that have or are being used by various agencies in the United States and other countries are presented. For the treaded-tire and 0.020-in. water-film thicknesses used in this study, macrotexture was found to have little effect on friction level, but did appreciably influence the percentage decrease in friction with speed.

•FRICTION properties of pavement surfaces have become factors of major importance to the overall traffic safety problem. Although friction measurements of the tire-pavement combination are considered acceptable for evaluating the skid-resistant properties of pavement surfaces, attempts are being made to characterize the skid-resistant properties in qualitative manners, such as macrotexture, drainage characteristics of the surface and aggregate size, shape, microtexture, and mineralogy. Most of these qualitative tests are not convenient survey measures; however, a basic understanding of the relative effects of the measured factors on pavement friction is a necessity in order to more fully understand the interaction at the tire-pavement interface and, thus, to enable the designer to understand the need for these desirable properties in the pavement surface.

Several researchers have indicated that the type and magnitude of texture are important characteristics of pavement surfaces with respect to friction properties (12, 13, 22, 23). Pavement-surface texture refers to the distribution and the geometrical configuration of the individual surface aggregates. There is not sufficient agreement among the various researchers to adopt a standard nomenclature for discussing textural parameters. However, general practice today favors the use of the terms (a) macroscopic texture (macrotexture) to refer to that part of the pavement surface as a whole or the large-scale texture caused by the size and shape of the surface aggregate and (b) microscopic texture (microtexture) to refer to the fine-scaled roughness contributed by individual small asperities on the individual aggregate particles.

Paper sponsored by Committee on Bituminous Aggregate Bases and presented at the 49th Annual Meeting.

The researchers have conflicting claims on the relative merits of macrotexture and microtexture. Some contend that a high level of macrotexture is essential to provide the pavement drainage at high speeds (24), whereas microtexture is the main texture contributor for a given friction level. Still others believe that a combination of both macrotexture and microtexture is most desirable (13, 21).

Kummer and Meyer (13) proposed the classification shown in Figure 1, which delineates the 2 roughness types that affect the friction of pavement surfaces. Although 5 surface types are classified, only 3 levels of macrotexture, i.e., smooth, fine, and coarse, are identified because types 2 and 3 and types 4 and 5 respectively are the same as far as macrotexture level is concerned. Thus, a given level of macrotexture as measured by the majority of existing test methods does not appear to adequately assess the degree of roundness or grittiness the individual aggregates possess. It is the authors' opinions that this fact is the main contributor to the low coefficients of correlation between friction parameters and macrotexture obtained in this study.

Macrotexture and microtexture respectively provide for gross surface drainage and subsequent puncturing of the water film. Another factor that acts in combination with macrotexture and microtexture is internal drainage of the pavement surface itself. Goodwin (19), Hutchinson (20), and Gallaway (21), among others, have postulated that high void content surfaces, porous pavements, or vesicular aggregates would provide internal escape paths for water under a tire and thus lessen hydrodynamic pressure buildup. This would result in better tire-gripping capability and increased traction, particularly at higher speeds, while decreasing the need of macrotexture for providing initial, gross drainage. Research directed toward measuring dynamic drainage capabilities of pavement surfaces is in the development stage (20). The authors believe that the combined effects of macrotexture and microtexture and internal drainage largely determine the friction levels of pavement surfaces. In this paper only the effects of macrotexture are examined.

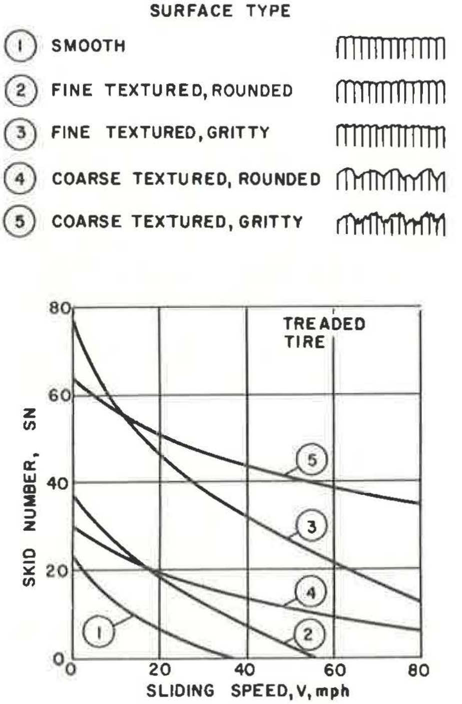


Figure 1. Classification of pavement surfaces according to their friction and drainage properties (13).

Various agencies in the United States and other countries are engaged in developing methods for measuring pavement-surface macrotexture in order to more fully evaluate its role in vehicle braking, cornering, and accelerating manuevers.

Methods that have been or are being used include the sand patch (1), NASA grease (2), drainage meter (3), foil piercing (4), linear traverse (5), stereophotographic (6), casting or molding (7), impression (8), centrifuge kerosene equivalent (9), wear and roughness meter (10), mineralogical studies and profilograph (11), and photo interpretation (12). Descriptions of 4 additional methods used in this research are given in the equipment discussion.

The phase of the research reported here had the following 3 objectives:

1. Analyze and compare different methods for measuring pavement surface macrotexture (both volumetric and mechanical roughness detector methods were used for assessing macrotexture levels);
2. Determine effects of macrotexture types and magnitudes on friction numbers and friction-speed gradients of various pavement surface types; and
3. Survey normal range of macrotexture existing on Texas pavements for use in a

subsequent study of the effects of variable macrotexture on water depth buildup for various levels of pavement cross slopes and rainfall intensities.

TEST EQUIPMENT

Skid Test Trailer

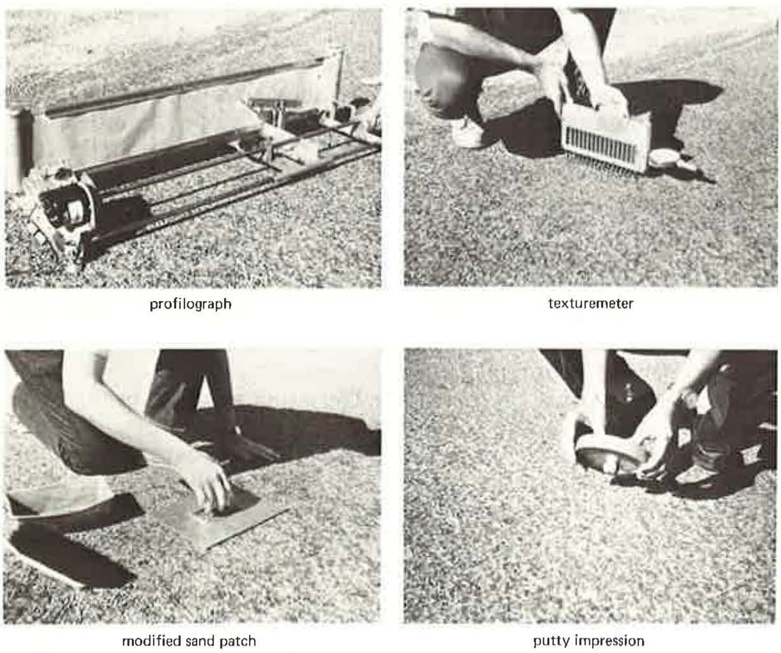
Figure 2. Texas Highway Department research skid trailer.

The friction measurements reported here were obtained with the Texas Highway Department research skid trailer that conforms substantially to ASTM standards and utilizes E-17 treaded tires inflated to 24 psi. The drag forces were measured with strain gages, and the self-watering system utilized a centrifugal pump that applied a water film approximately 0.020 in. in thickness to the pavement surface. The development and calibration of the trailer may be reviewed in an earlier research report (14). Figure 2 shows the trailer under test conditions.

Macrotexture Measurements

Four methods were used to obtain 5 measures of macrotexture. Equipment used for these are shown in Figure 3. Two measures of average peak height and 2 measures of average texture depth were obtained, and these measurements were reduced to equivalent units. Also, one measure of accumulative peak height was obtained. Information on the methods is given in Table 1, and each is briefly described in the following.

Profilograph—The instrument used for this test was developed by the Texas Highway Department (15). It is designed to scribe a magnified profile of the surface texture as a probe is drawn across the surface. That is, the probe is placed on the pavement



profilograph

texturemeter

modified sand patch

putty impression

Figure 3. Equipment used for macrotexture tests.

TABLE 1

METHODS USED FOR MACROTEXTURE TESTS

Method	Measure (in.)
Profilograph	Average peak height
Profilograph	Accumulative peak height
Texturemeter	Average peak height
Modified sand patch	Average texture depth
Putty impression	Average texture depth

surface and, as the probe is drawn over the surface irregularities, the vertical movement of the probe is magnified through a linkage system. The probe and linkage system are attached to a carriage that is forced to move in a horizontal manner parallel to the pavement surface by a framework with adjustable legs. The vertical and horizontal movement results in a duplicated (but magnified) texture profile that is scribed on a chart. Average peak height can then be determined. Also, the upward vertical excursions are recorded on a counter of which the counter reading, at any time, is the cumulative vertical peak heights (referenced to a median line in the surface) of the texture through the length traversed by the probe.

Texturemeter—The instrument, developed at the Texas Transportation Institute (16) and used previously for macrotexture tests (17), consists essentially of a series of evenly spaced, parallel rods mounted in a frame. The rods can be moved vertically, independently of one another, against spring pressure. At either end of the series of movable rods is a fixed rod rigidly attached to the frame. Each movable rod is pierced by a hole through which passes a taut string, one end of which is fixed to the frame and the other to the spring-loaded stem of a 0.001-in. dial gage mounted on the frame. When the instrument is in use, the rods are held in a vertical position with their ends resting against the pavement surface. If the surface is smooth, the string will form a straight line and the dial will read zero. Any irregularities in the surface will cause the string to form a zig-zag line and will result in a dial reading; the coarser the pavement texture is, the larger the dial reading will be. Average peak height can be calculated from the dial reading. The readings given by an instrument of this kind are affected by the size and spacing of the rods and the distance spanned by these rods. For the texturemeter used, the rods are spaced at $\frac{5}{8}$ in., and the instrument spans a distance of 10 in. between fixed supports.

Modified Sand Patch—This method was modified slightly from that developed by the British (1). Equipment consists of (a) a 6.15- by 4.60-in. rectangular metal plate $\frac{1}{8}$ in. thick with a 4.35- by 2.90-in. center hole and a 2-in. wide $\frac{1}{16}$-in. thick collar or free board; (b) 100 grams of a fine-grained sand; and (c) a 4-in. long straightedge. The technique involves determining the increase in the volume of sand required to fill the metal plate cavity when placed on a textured surface above the volume required to fill the cavity when placed on a nontextured surface. If the plate is placed on a textured surface, the bottom of the plate will rest on the upper aggregate asperities. The more irregular the surface texture is, the larger the resulting weight of sand required to fill the cavity will be. The average texture depth is defined as the ratio of the increased volume of sand to the area of the patch.

Putty Impression—This method was initially developed as a means of providing surface texture correction factors for nuclear density measurements (18). Equipment consists of (a) a 6-in. wide by 1-in. thick metal plate with a 4-in. wide by $\frac{1}{16}$-in. deep recess machined into one side, and (b) a 15.90-gram ball of silicone putty. When placed on a smooth surface, 15.90 grams of putty will smooth out to a circle 4 in. wide by $\frac{1}{16}$ in. deep, thus completely filling the recess. The silicone putty is formed into an approximate sphere and placed on the pavement surface. The recess in the plate is centered over the putty, and the plate is pressed down in firm contact with the road surface. The more irregular the surface texture is (the higher the macrotexture), the smaller the resulting putty diameter will be because more material is required to fill the surface texture. Average texture depth, based on volume per unit area, is calculated from an average of 4 diameter measurements.

SURFACE TYPES

Forty-one pavement surfaces were tested including 21 hot-mix asphalt concrete surfaces, 9 portland cement concrete surfaces, 9 surface treatments, and 2 seal coats.

The term surface as used in this paper is defined as a section of pavement on which the wearing course is essentially identical over the entire length under study. These test surfaces were selected with regard to level of service, degree of polish, and traffic volume. In addition, it was planned for the test sample to include at least 10 surfaces from each major service category of the Texas Highway System. The array of surface types selected included the various configurations of mineralogical types and aggregate sizes commonly used in Texas. Tests were also made on new surface designs, which are not widely used at present but for which increased use is envisioned for the future. Information and summary data for the surfaces are given in Tables 2 and 3. Skid number-speed curves are shown in Figures 4, 5, and 6. Differences in skid numbers found on the 41 pavements are evident from these data. The surfaces were classified with respect to the type of coarse aggregate contained. Lightweight aggregate implies an expanded clay or shale.

TEST PROCEDURE

A series of 20-, 40-, and 60-mph skid tests were conducted at 4 locations on each test surface. Ten texture measurements were taken at each location for a total of 40 measurements per surface. All measurements were made in the outer wheelpath.

Average skid numbers at 20, 40, and 60 mph respectively with appropriate temperature corrections were calculated for each test surface. In addition, for use in subsequent comparisons, average skid numbers between 20 and 60 mph were calculated. Provided abrupt slope changes in the skid number-speed curve do not occur, the average skid number is very nearly equal to the skid number at 40 mph. Calculations are shown in Figure 7.

Gradients (denoted by G) of the skid number-speed curve between 20 and 60 mph and between 20 and 40 mph were calculated. These have been used in previous reports. In addition, in order to reflect the relative position of the curve, percentage gradients were calculated. Curves of a given gradient positioned low on the graph would have higher percentage gradients than curves with the same gradient positioned high on the graph. Thus,

TABLE 2

CLASSIFICATION OF TEST SURFACES

Surface and Aggregate	Number Tested
Hot-mix asphalt concrete	21
Lignite boiler slag aggregate	3
Rounded siliceous gravel	4
Crushed limestone aggregate	4
Crushed siliceous gravel	4
Crushed sandstone aggregate	3
Lightweight aggregate	3
Portland cement concrete	9
Rounded siliceous gravel	5
Crushed limestone aggregate	3
Rounded siliceous gravel and crushed sandstone aggregate mixture	1
Surface treatment and seal coat	11
Rounded siliceous gravel	2
Crushed limestone aggregate	4
Lightweight aggregate	3
Flushed bituminous seal	2

TABLE 3

SKID NUMBER AND MACROTEXTURE VALUES

Surface	Number Tested	Skid Number Range at 40 mph	Macrotexture Range (in.)
Hot-mix asphalt concrete	21	29-59	0.01-0.04
Portland cement concrete	9	36-45	0.01-0.04
Surface treatment	9	29-65	0.02-0.07
Seal coat	2	18-27	0.00-0.01

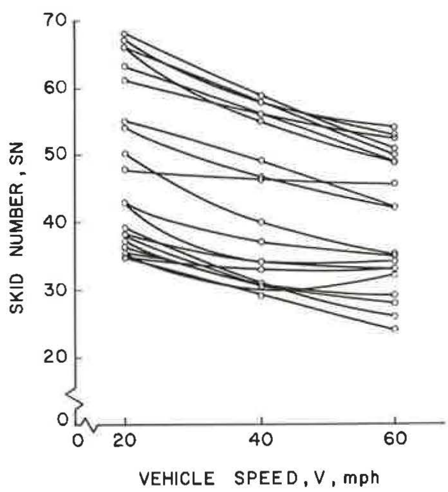


Figure 4. Skid number-speed relationships for hot-mix asphalt concrete surfaces.

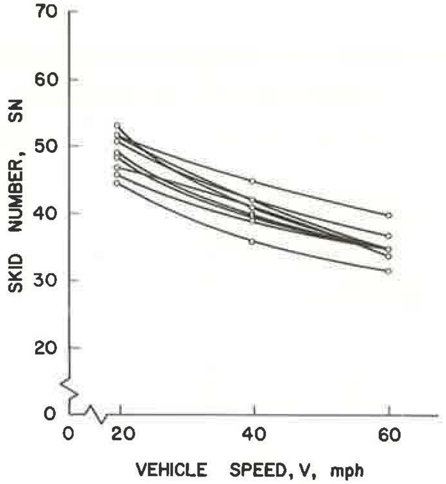


Figure 5. Skid number-speed relationships for portland cement concrete surfaces.

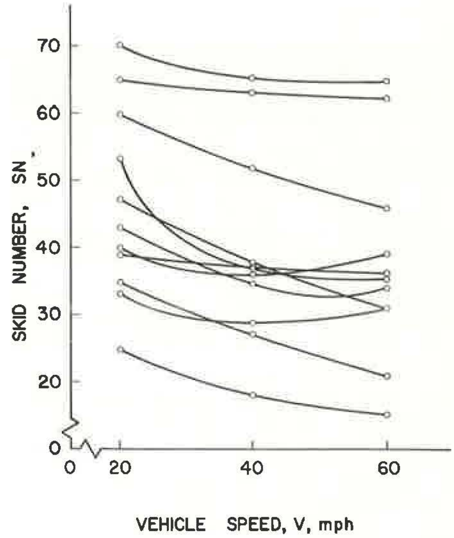
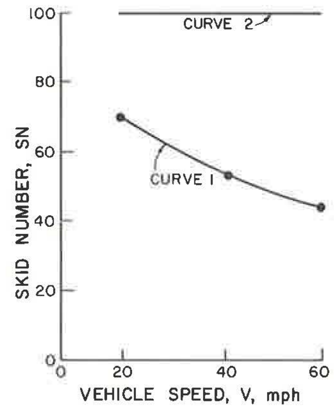


Figure 6. Skid number-speed relationships for surface treatments and seal coats.



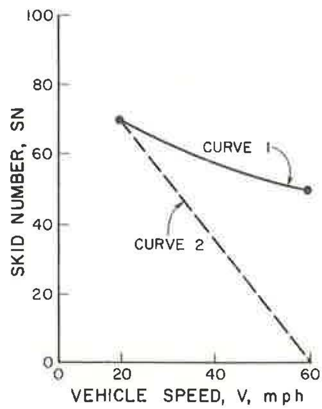
AVERAGE SKID NUMBER $A_V SN$ =

$$\frac{\text{AREA UNDER CURVE 1 } (A_1)}{\text{AREA UNDER CURVE 2 } (A_2)} \times 100$$

$$A_V SN_{20-60} \approx SN_{40}$$

NOTE: A_2 IS CONSTANT = 4000 UNITS

Figure 7. Average skid number calculation.



$$\text{GRADIENT } (G) = \frac{SN_{20} - SN_{60}}{40}$$

$$\text{GRADIENT } (G_2) = \frac{SN_{20} - 0}{40}$$

PERCENTAGE GRADIENT (PG) =

$$\frac{G}{G_2} \times 100 = \frac{SN_{20} - SN_{60}}{SN_{20}} \times 100$$

Figure 8. Gradient and percentage gradient calculations.

percentage gradient is defined as the percentage of the gradient, obtained under test conditions, to a theoretical gradient if the skid number at the higher speed were zero. Calculations are shown in Figure 8.

DISCUSSION OF RESULTS

Statistical analyses were conducted to determine correlation coefficients, coefficients of determination, and regression lines for the comparisons established in this study. Results are given in Tables 4 through 7. Table 8 gives detailed macrotexture and friction values.

TABLE 4

STATISTICAL CORRELATION OF MACROTEXTURE TEST METHODS

No.	Variables Y	Variables X	Regression Line	Correlation Coefficient	Coefficient of Determination	Standard Deviation
1	TDS	TDP	Y = -0.00 + 1.41 X	0.93	0.86	0.009
2	TDS	APHP	Y = 0.02 + 0.03 X	0.86	0.74	0.012
3	TDS	PHP	Y = -0.01 + 1.80 X	0.81	0.66	0.014
4	TDS	PHTM	Y = 0.00 + 1.75 X	0.86	0.74	0.012
5	TDP	APHP	Y = 0.02 + 0.02 X	0.78	0.61	0.009
6	TDP	PHP	Y = -0.00 + 1.11 X	0.77	0.59	0.010
7	TDP	PHTM	Y = 0.01 + 1.08 X	0.81	0.65	0.009
8	APHP	PHP	Y = -1.01 + 68.64 X	0.92	0.85	0.306
9	APHP	PHTM	Y = -0.42 + 63.30 X	0.92	0.85	0.307
10	PHP	PHTM	Y = 0.01 + 0.76 X	0.83	0.68	0.006

Note: TDS = average texture depth, sand method; TDP = average texture depth, putty method; APHP = accumulative peak height, profilograph method; PHP = average peak height, profilograph method; and PHTM = average peak height, texturemeter method.

TABLE 5

STATISTICAL CORRELATION OF SKID NUMBER AND MACROTEXTURE

No.	Variables Y	Variables X	Regression Line	Correlation Coefficient	Coefficient of Determination	Standard Deviation
1	SN_{20}	TDS	Y = 48.35 + 13.18 X	0.03	0.00	11.79
2	SN_{20}	TDP	Y = 45.09 + 140.37 X	0.18	0.03	11.60
3	SN_{20}	APHP	Y = 47.54 + 1.87 X	0.12	0.02	11.70
4	SN_{20}	PHP	Y = 44.23 + 186.81 X	0.17	0.03	11.63
5	SN_{20}	PHTM	Y = 47.56 + 71.59 X	0.07	0.00	11.77
6	SN_{40}	TDS	Y = 40.09 + 46.58 X	0.10	0.01	11.26
7	SN_{40}	TDP	Y = 37.01 + 175.08 X	0.24	0.06	10.99
8	SN_{40}	APHP	Y = 39.17 + 3.67 X	0.25	0.06	10.94
9	SN_{40}	PHP	Y = 33.87 + 317.44 X	0.30	0.09	10.81
10	SN_{40}	PHTM	Y = 38.51 + 181.39 X	0.18	0.03	11.12
11	SN_{60}	TDS	Y = 34.37 + 110.84 X	0.24	0.06	10.43
12	SN_{60}	TDP	Y = 31.44 + 250.00 X	0.35	0.13	10.04
13	SN_{60}	APHP	Y = 34.26 + 5.63 X	0.41	0.17	9.79
14	SN_{60}	PHP	Y = 26.36 + 477.12 X	0.47	0.22	9.49
15	SN_{60}	PHTM	Y = 32.73 + 307.97 X	0.33	0.11	10.15
16	ASN	TDS	Y = 40.74 + 49.63 X	0.10	0.01	11.03
17	ASN	TDP	Y = 37.65 + 179.60 X	0.25	0.06	10.75
18	ASN	APHP	Y = 39.97 + 3.60 X	0.25	0.06	10.73
19	ASN	PHP	Y = 34.64 + 316.77 X	0.30	0.09	10.58
20	ASN	PHTM	Y = 39.26 + 181.42 X	0.19	0.03	10.90

Note: SN = skid number, subscript indicating speed in mph; ASN = average skid number between 20 and 60 mph; TDS = average texture depth, sand method; TDP = average texture depth, putty method; APHP = accumulative peak height, profilograph method; PHP = average peak height, profilograph method; and PHTM = average peak height, texturemeter method.

TABLE 6

STATISTICAL CORRELATION OF GRADIENT AND MACROTEXTURE

No.	Variables Y	X	Regression Line	Correlation Coefficient	Coefficient of Determination	Standard Deviation
1	G_{20-40}	TDS	Y = 0.41 - 1.67 X	-0.27	0.07	0.140
2	G_{20-40}	TDP	Y = 0.40 - 1.74 X	-0.18	0.03	0.143
3	G_{20-40}	APHP	Y = 0.42 - 0.09 X	-0.48	0.23	0.127
4	G_{20-40}	PHP	Y = 0.52 - 6.53 X	-0.47	0.22	0.128
5	G_{20-40}	PHTM	Y = 0.45 - 5.50 X	-0.43	0.19	0.131
6	G_{20-40}	ln TDS	Y = 0.31 - 0.01 ln X	-0.08	0.01	0.145
7	G_{20-40}	ln TDP	Y = 0.27 - 0.02 ln X	-0.11	0.01	0.145
8	G_{20-40}	ln APHP	Y = 0.33 - 0.02 ln X	-0.24	0.06	0.141
9	G_{20-40}	ln PHP	Y = -0.09 - 0.12 ln X	-0.35	0.13	0.136
10	G_{20-40}	ln PHTM	Y = 0.32 - 0.01 ln X	-0.07	0.01	0.145
11	G_{20-60}	TDS	Y = 0.35 - 2.44 X	-0.44	0.20	0.115
12	G_{20-60}	TDP	Y = 0.34 - 2.75 X	-0.33	0.11	0.121
13	G_{20-60}	APHP	Y = 0.33 - 0.09 X	-0.57	0.33	0.105
14	G_{20-60}	PHP	Y = 0.45 - 7.24 X	-0.59	0.35	0.103
15	G_{20-60}	PHTM	Y = 0.37 - 5.91 X	-0.53	0.28	0.109
16	G_{20-60}	ln TDS	Y = 0.16 - 0.03 ln X	-0.22	0.05	0.125
17	G_{20-60}	ln TDP	Y = 0.12 - 0.04 ln X	-0.23	0.05	0.125
18	G_{20-60}	ln APHP	Y = 0.24 - 0.03 ln X	-0.33	0.11	0.121
19	G_{20-60}	ln PHP	Y = - 0.27 - 0.14 ln X	-0.48	0.23	0.112
20	G_{20-60}	ln PHTM	Y = 0.21 - 0.01 ln X	-0.15	0.02	0.127

Note: G = gradient (slope) of the friction-speed curve, subscript indicating speed range in mph; TDS = average texture depth, sand method; TDP = average texture depth, putty method; APHP = accumulative peak height, profilograph method; PHP = average peak height, profilograph method; and PHTM = average peak height, texturemeter method.

TABLE 7

STATISTICAL CORRELATION OF PERCENTAGE GRADIENT AND MACROTEXTURE

No.	Variables Y	X	Regression Line	Correlation Coefficient	Coefficient of Determination	Standard Deviation
1	PG_{20-40}	TDS	Y = 17.7 - 81.0 X	-0.31	0.09	5.77
2	PG_{20-40}	TDP	Y = 18.3 - 122.3 X	-0.31	0.09	5.77
3	PG_{20-40}	APHP	Y = 17.9 - 4.2 X	-0.54	0.30	5.09
4	PG_{20-40}	PHP	Y = 22.9 - 321.4 X	-0.56	0.31	5.04
5	PG_{20-40}	PHTM	Y = 19.3 - 249.0 X	-0.47	0.22	5.36
6	PG_{20-40}	ln TDS	Y = 9.6 - 1.5 ln X	-0.21	0.05	5.93
7	PG_{20-40}	ln TDP	Y = 3.1 - 3.1 ln X	-0.37	0.14	5.63
8	PG_{20-40}	ln APHP	Y = 13.0 - 1.9 ln X	-0.45	0.20	5.42
9	PG_{20-40}	ln PHP	Y = -10.7 - 6.8 ln X	-0.49	0.24	5.31
10	PG_{20-40}	ln PHTM	Y = 12.1 - 0.7 ln X	-0.14	0.02	6.01
11	PG_{20-60}	TDS	Y = 29.6 - 225.1 X	-0.53	0.28	8.44
12	PG_{20-60}	TDP	Y = 30.1 - 301.7 X	-0.46	0.21	8.80
13	PG_{20-60}	APHP	Y = 28.0 - 8.7 X	-0.69	0.47	7.21
14	PG_{20-60}	PHP	Y = 38.8 - 680.3 X	-0.72	0.52	6.88
15	PG_{20-60}	PHTM	Y = 31.5 - 545. 7 X	-0.63	0.39	7.74
16	PG_{20-60}	ln TDS	Y = 7.2 - 4.0 ln X	-0.36	0.13	9.27
17	PG_{20-60}	ln TDP	Y = 2.6 - 6.5 ln X	-0.47	0.22	8.75
18	PG_{20-60}	ln APH	Y = 17.9 - 3.8 ln X	-0.56	0.31	8.23
19	PG_{20-60}	ln PHP	Y = -33.7 - 14.7 ln X	-0.64	0.41	7.60
20	PG_{20-60}	ln PHTM	Y = 14.2 - 1.8 ln X	-0.24	0.06	9.65

Note: PG = percentage gradient of the friction-speed curve, subscript indicating speed range in mph; TDS = average texture depth, sand method; TDP = average texture depth, putty method; APHP = accumulative peak height, profilograph method; PHP = average peak height, profilograph method; and PHTM = average peak height, texturemeter method.

TABLE 8

MACROTEXTURE AND FRICTION VALUES

Surface and Aggregate	Number Tested	Macrotexture: Profilograph Avg. Peak Height[a] (in.)	Macrotexture: Profilograph Accum. Peak Height (in.)	Macrotexture: Texture-meter Avg. Peak Height[a] (in.)	Macrotexture: Modified Sand Patch Avg. Texture Depth[a] (in.)	Macrotexture: Putty Impression Avg. Texture Depth[a] (in.)	Friction: Skid Number 20 mph	Friction: Skid Number 40 mph	Friction: Skid Number 60 mph	Friction: Gradient 20-40 mph	Friction: Gradient 20-60 mph	Friction: Percentage Gradient 20-40 mph	Friction: Percentage Gradient 20-60 mph
Hot-mix asphalt concrete													
Lignite boiler slag aggregate	3	0.0184	0.023	0.0021	0.0078	0.0079	47	41	37	0.30	0.24	13	21
Rounded siliceous gravel	4	0.0252	0.741	0.0215	0.0317	0.0276	44[b]	39[b]	37[b]	0.25[b]	0.18[b]	11[b]	16[b]
Crushed limestone aggregate	4	0.0177	0.308	0.0123	0.0205	0.0175	49[b]	40[b]	36[b]	0.45[b]	0.31[b]	18[b]	26[b]
Crushed siliceous gravel	4	0.0227	0.589	0.0179	0.0308	0.0276	47[b]	41[b]	39[b]	0.30[b]	0.21[b]	13[b]	18[b]
Crushed sandstone aggregate	3	0.0215	0.299	0.0142	0.0159	0.0212	68	60	56	0.40	0.29	12	17
Open-graded lightweight aggregate	3	0.0262	1.107	0.0264	0.0406	0.0324	64	55	49	0.45	0.38	14	23
Average	21	0.0219	0.516	0.0159	0.0250	0.0226	52	45	42	0.35	0.26	13	20
Portland cement concrete													
Rounded siliceous gravel	5	0.0210	0.239	0.0095	0.0230	0.0202	52	44	39	0.40	0.35	15	26
Crushed limestone aggregate	3	0.0188	0.128	0.0074	0.0195	0.0170	51	42	36	0.45	0.37	18	29
Rounded siliceous gravel and crushed sandstone aggregate	1	0.0203	0.355	0.0140	0.0535	0.0308	52	43	38	0.45	0.35	17	27
Average	9	0.0202	0.215	0.0093	0.0252	0.0203	52	43	38	0.45	0.36	17	27
Surface treatment													
Rounded siliceous gravel	2	0.0463	2.527	0.0425	0.0794	0.0464	42	39	40	0.15	0.06	7	5
Crushed limestone aggregate	4	0.0267	0.879	0.0226	0.0569	0.0450	46	37	35	0.45	0.30	20	24
Lightweight aggregate	3	0.0425	1.943	0.0295	0.0649	0.0470	67	62	60	0.25	0.19	7	11
Average	9	0.0363	1.600	0.0293	0.0646	0.0460	52	46	44	0.30	0.21	12	15
Flushed bituminous seal	2	0.0154	0.035	0.0094	0.0049	0.0030	33	25	21	0.40	0.29	24	36
Average	2	0.0154	0.035	0.0094	0.0049	0.0030	33	25	21	0.40	0.29	24	36

[a]These are comparable values even though the descriptive terms differ.

[b]Includes data from 1 test surface at Annex that increases friction averages.

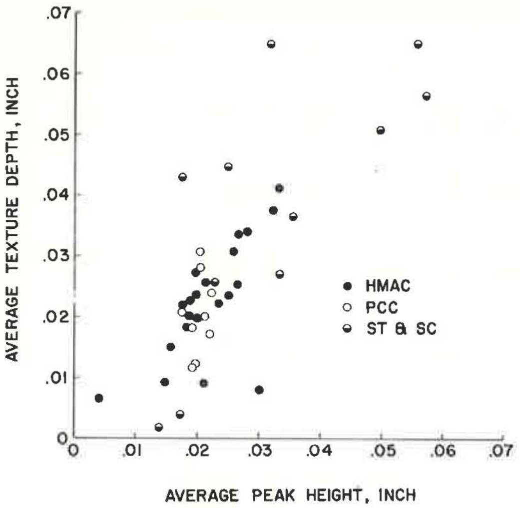


Figure 9. Putty impression versus profilograph methods.

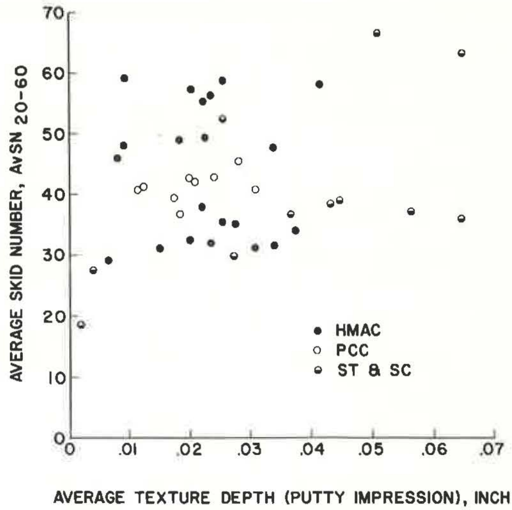


Figure 10. Average skid number versus macrotexture.

Comparison of Macrotexture Test Methods

Statistical correlation of the 5 macrotexture measures obtained on the 41 surfaces are given in Table 4. The relationships indicate a fairly high degree of correlation. A typical plot of one relationship is shown in Figure 9. The most diverse data scatter was obtained at the extremities of the texture levels.

Comparison of Skid Number and Macrotexture

Statistical correlation of skid numbers with macrotexture measures obtained on the 41 surfaces are given in Table 5. Correlation coefficients were extremely low for all speed levels, but the relative magnitudes increased with higher speeds. Textures obtained with the profilograph consistantly correlated better with skid numbers. A typical plot of one relationship is shown in Figure 10. A slight trend is noticeable.

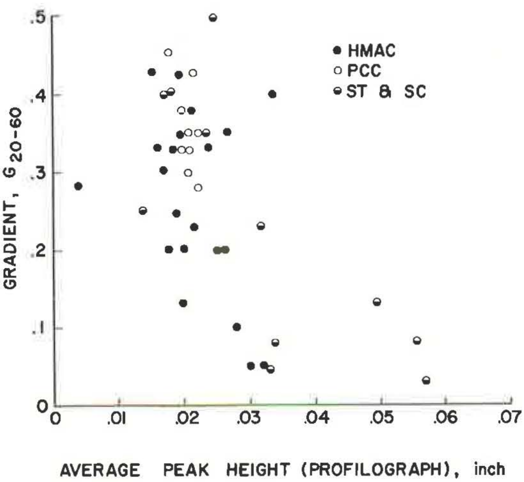


Figure 11. Gradient versus macrotexture.

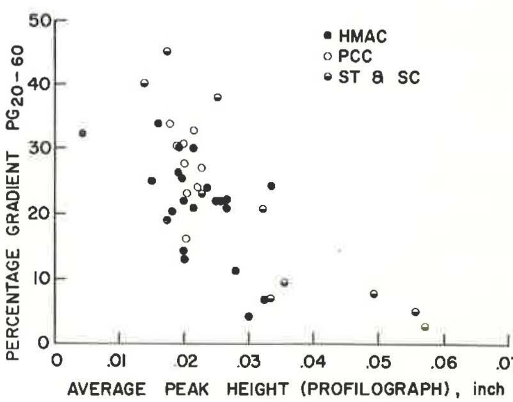


Figure 12. Percentage gradient versus macrotexture.

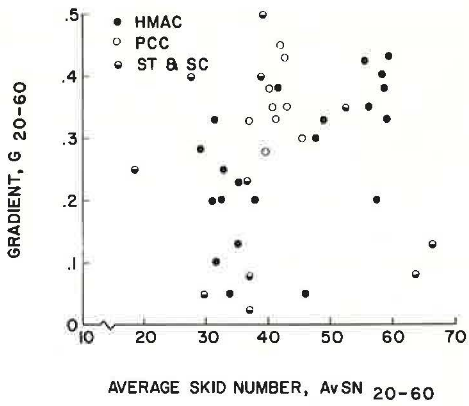


Figure 13. Gradient versus average skid number.

Comparison of Gradient and Macrotexture

Statistical correlation of friction-speed gradients obtained from 20-40 mph and from 20-60 mph with macrotexture measures obtained on the 41 surfaces are given in Table 6. Correlation coefficients were fairly low, particularly for the logarithmic relationships. Gradient computations from 20-60 mph resulted in higher coefficients than those from 20-40 mph. Also, the mechanical roughness detector instruments (profilograph and texturemeter) gave higher coefficients than the volumetric measures (putty and sand). A typical plot is shown in Figure 11.

Comparison of Percentage Gradient and Macrotexture

Statistical correlation of friction-speed percentage gradients obtained from 20-40 mph and from 20-60 mph with macrotexture measures obtained on the 41 surfaces are given in Table 7. Relative trends were the same as those for corresponding gradient comparisons; however, magnitudes in each case were greater, indicating better correlation. Again, as was evident from the gradient comparisons presented previously, macrotexture effects on friction increased with higher speeds. A typical plot is shown in Figure 12.

Comparison of Gradient and Skid Number

Friction-speed gradients and average skid numbers from 20-60 mph for the 41 pavements are shown in Figure 13. A relationship does not exist.

CONCLUSIONS

Based on the specific test conditions, equipment limitations, and pavement surface types utilized in this study, the following general conclusions appear to be warranted.

Pavement surface macrotexture measurement may be affected by any one of several different methods, and the units of such measurements may be the same; however, the contribution of such macrotexture to the frictional drag of a given tire-pavement system is not easily evaluated. A primary reason for the difficulty is that, for any selected method of macrotexture measurement, there are numerous possible combinations of pavement surface rugosity that will yield similar texture values. It may be further stated that, for given equipment and environmental conditions, pavements with approximately equal macrotexture values may very well have widely different skid numbers where such numbers are obtained by methods currently listed by Committee E-17 in ASTM Standards.

A primary function of macrotexture is water drainage from the tire-pavement interface. This drainage becomes more important as (a) vehicle speed increases, (b) tire tread depth decreases, and (c) water depth increases. In addition, macrotexture causes tire wrinkling, and this texture acts as an energy absorber in rolling, slipping, and skidding. This action brings into play the hysteresis properties of the tire rubber. The energy absorption caused by the hysteresis of the tire rubber increases with vehicle speed.

The study further shows that, of the means investigated, the profilograph method is the most convenient way to measure macrotexture; however, statistically, a rather poor correlation between macrotexture and skid resistance was shown. One explanation for this finding is that macrotexture of the aggregate was disregarded, and a second reason is quite likely lodged in the limited variation in water-film thickness studied.

The range of macrotexture found on the pavements included in the study covered, in reasonable fashion, the range one might encounter across the entire United States. Of the 41 pavements tested, texture values ranged from 0.0+ to about 0.07 in. A pavement with a macrotexture value higher than about 0.10 would be noisy and, if composed of sharp textured gritty particles, would create excessive tire wear without offsetting benefits in vehicle control.

As already indicated, water-film thickness and tire-tread depth are important parameters affecting the impact of a given macrotexture on frictional drag. A study of the relative magnitudes of these effects is under way, and definitive results will be available within a matter of months.

Drainage of the tire-pavement interface may be effected into the pavement as well as between the tire and the pavement. Open-graded, lightweight aggregate asphalt concrete has been designed and placed into service, and these pavements appear to offer important bonuses in this area. Three such pavements were included in this study. This design would offer partial solutions to drainage problems in transitions from tangents to superelevated curves on 2-lane facilities and on compound curves.

SUMMARY

1. The 4 methods used to evaluate pavement surface macrotexture provide acceptable data, and texture values obtained by the different methods compared favorably.

2. The profilograph method for measurement of macrotexture is preferred because of its simplicity, reproducibility, and better correlation with friction parameters. However, statistically, even results obtained with the profilograph do not relate favorably with friction parameters.

3. Macrotexture was found to range from 0.00 to 0.07 in. on a random sample of 41 Texas highways. The larger values were associated with surface treatments composed almost entirely of aggregate; whereas, the smaller values were associated with "flushed" seals. The majority of the surfaces were in the 0.015- to 0.035-in. range, which included most of the hot-mix asphalt concrete and portland cement concrete pavements.

4. The effect of the aggregate microtexture is included in many of the measurements made in the study, but the magnitude of the effect remains unknown. A clear understanding of the total problem hinges, at least in part, on the magnitude of the effect of microtexture.

(The following summary statements are predicated on the existence of a reasonably constant water-film thickness of 0.02 in. for all surfaces that were tested.)

5. No correlation was found between 20-, 40-, and 60-mph skid numbers and macrotexture. Macrotexture effects accounted for a maximum of 3 percent at the variation in skid numbers at 20 mph, 9 percent at 40 mph, and 22 percent at 60 mph.

6. Poor correlation was found between gradients of the friction-speed curve and macrotexture. A maximum of 23 percent at the variations in 20- to 40-mph gradients was explained by macrotexture effects, whereas 35 percent of the variations in 20- to 60-mph gradients was explained.

7. A fair correlation was found between percentage gradients of the friction-speed curve and macrotexture. In these cases maximums of 31 and 52 percents of the variations in 20- to 40- and 20- to 60-mph percentage gradients respectively were explained by macrotexture effects.

8. A relationship between gradient and skid number was not obtained.

9. The existence of a high macrotexture level as measured by the 4 methods does not ensure a high coefficient of friction.

10. The existence of extremely large-scale macrotexture (>0.035 in.) ensures a relatively flat friction-speed gradient; however, macrotexture (<0.035 in.) does not ensure a flat friction-speed gradient.

11. Water depth on the pavement surface was held reasonably constant in the study; so, the effect of varying this factor was not investigated. Skid numbers are largely affected by microtexture and macrotexture on the surface and internal drainage into the surface as well as pavement water depth and vehicle speed.

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Effect of Studded Tires on Aggregate and Related Effects on Skid Resistance

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•DURING the past 2 or 3 years in Minnesota, we have witnessed a startling increase in the amount of surface wear in the wheelpaths on both bituminous and portland cement concrete pavements. Prior to this period, pronounced surface abrasion of this type occurred usually in isolated instances only. When an occasional bituminous pavement suffered such attrition, it generally occurred fairly early in the life of the surface and was likely to be related to some accountable deficiency such as unfavorable construction conditions, or possibly marginal materials or mixture design for the particular circumstances or traffic demands involved.

The wear related to studded-tire effects is, however, different in several respects. It has affected pavements of the highest quality in Minnesota, including some that had been in service for several years before being subjected to studded-tire traffic and others that were exposed to studs in their initial year. The phenomenon has developed during the past 4 years, the period in which studded tires have been legally permitted during the winter months on Minnesota's highways. Because the severe wear has developed concurrently with the studded-tire usage, it is quite natural that the highway engineers generally blame the wear on the studs. There are others, however, who ascribe the attrition to increased use of salts and sand, contending that the chemicals attack the pavement materials and that they, together with abrasive sand, make the pavement surfaces readily susceptible to wear from the studs. This argument, however, overlooks the fact that salt and sand have been used freely for many years with little or no damage to pavements.

This brief history of studded tires in Minnesota will provide some background. In 1964 when they first began to appear, studs were ruled illegal under the then-existing law. The highway department initiated some limited field-driving tests with studs, and these continued during winter intervals into January 1967. Results indicated definite possibility of objectionable wear, but the tests were not extensive enough to convince the lawmakers that studs should not be permitted in the interest of safety. The 1965 legislature authorized a 2-year "trial" period for the winters 1965-66 and 1966-67, after which the 1967 and the 1969 legislatures again authorized additional 2-season extensions. Therefore, we are now in the fifth winter of studded-tire traffic. Use of studs has increased phenomenally in this period.

During the first winter, the number of studded tires used in the metropolitan area was very low; only about 3.8 percent of the cars were so equipped on the rear wheels. During the second winter, the proportion of cars with studs increased to about 9 percent. During the third winter, the number increased substantially to 23 percent, and by the 1968-69 winter it had reached almost 32 percent. These proportions are based on surveys conducted on randomly selected parking lots and street parking. It is estimated that during the 1969-70 winter the proportion reached about 40 percent, and that 90 percent of all snow tires sold were studded.

While authorizing the latest time extension for studs, the legislature also directed the Commissioner of Highways to make an in-depth study of the studded-tire problem to determine (a) the damage, if any, to pavements that results from the use of metal tire studs, salt de-icing materials, and other materials of a chemical or a physical

Paper sponsored by Committee on Bituminous Aggregate Bases and presented at the 49th Annual Meeting.

nature; (b) whether such damage, if any, could be reduced by making changes in asphalts, concrete aggregates, or other highway surface materials; and (c) the effects, if any, that discontinuing the use of studded tires will have on highway safety.

Studies in these areas are being conducted (two under separate contracts with independent research agencies and others through our own continuing field investigations) to develop information to report to the legislature in 1971. Unfortunately, at this stage not enough data have been developed to provide significant quantitative evidence in response to the questions imposed. However, our field observations during the past 4 years when studs have been used during the winters have revealed some interesting facts.

The amount of pavement wear that developed in the first 2 winters was relatively minor, apparently because of the small number of cars equipped with studs. As the proportion of users increased rapidly in the third winter, the wear became pronounced. By the end of the fourth winter, the wear had become severe on some roads, reaching a depth as much as $\frac{1}{4}$ in. in the wheelpaths on both bituminous and portland cement concrete.

Wear measurements made at a limited number of test points at the beginning and end of winter seasons indicated that the amount of wear occurring during the summer months was much lower, in some cases insignificant, compared with that experienced in the winter months.

Based on our earliest field-driving tests with studs, which were necessarily very abbreviated as compared with highway traffic, it had been anticipated that the most severe abrasion might develop at locations of channelized traffic with concentrated stopping and starting actions, such as at semaphore-controlled intersections. This appears now to be less critical than first expected, apparently because of the lower speeds, acceleration, and volumes.

The most severe wear has developed on high-speed, high-volume roads such as the Interstate and other freeways and expressways within and around the metropolitan area. The rate of pavement wear thus appears to be definitely related to both speed and number of stud applications. No general quantitative relationship has yet been established, but, as an example, at one location where the wear depression was about $\frac{1}{4}$ in. it was estimated from traffic data that there had been about 1.6 million stud applications during the 4 winters of exposure.

In contrast, on rural roads having low traffic volumes the wear is slow to develop, even though travel speeds are high. On the other hand, on urban streets with high traffic density the wear rate is also low, evidently because of the slower speeds. However, where there is a concentration of fast acceleration or heavy braking action, such as on entrance or exit ramps to or from freeways, the wear is quite pronounced.

The manner in which the abrasive wear affects pavement surfaces appears to be substantially the same for both bituminous and portland cement concrete pavements. As the surface film or coating of either asphalt or cement mortar wears away, the coarse aggregate particles are gradually exposed to view so that the first clearly visible evidence of significant wear is the mosaic-like appearance of the surface.

As the wear progresses the matrix is eroded from between the harder coarse aggregate particles. It is noticeable that, where the coarse aggregate either is crushed rock consisting entirely of hard wear-resistant igneous particles or is crushed gravel containing a high proportion of igneous pebbles, these hard particles are left protruding above the surrounding matrix. This produces a knobby, rough-textured surface that is readily discernible, visually and by sound and feel, as a car travels over the pavement. The area affected in the wheelpaths is generally approximately $2\frac{1}{2}$ ft wide for each wheelpath.

Coarse aggregate consisting of relatively soft limestone tends to wear down more or less with the removal of the matrix. The result is that the wheelpath surface is not as rough and may not appear to be as badly abraded as is the case with the harder rock materials or gravel aggregates of mixed composition. The limestone coarse aggregate may even at times wear somewhat more rapidly than the matrix so that the surfaces of the coarse aggregate particles are actually slightly depressed below the matrix. This situation would be dependent largely on the composition and character of the matrix, including the quality of the sand and the proportion of binder material in the mixture.

These observations with reference to the aggregates are only generalizations at this point and need verification before any firm conclusions may be drawn. It is apparent, however, that the kind of aggregates used, both coarse and fine, may have significant influence on the rate of pavement wear. It is expected that further light on this will be gained from the laboratory project and the related field studies that have been initiated.

The development of wheelpath wear produces, in effect, shallow surface ruts that may possibly have some effect on the skid characteristics of the pavement. Our plans to make skid test measurements during the fall months failed to materialize because of the late delivery of the 2-wheeled trailer type of skid tester that had been purchased by the highway department. However, a limited number of measurements were made with the trailer test unit of an outside agency.

At a number of locations, readings were taken both within the wheelpath and outside the worn area of the wheelpath. In nearly all cases, the skid numbers obtained were within 2 or 3 points of one another for companion readings inside and outside the wheelpath. At most of the test points where the comparison readings were taken, a slightly lower reading (2 to 3 skid numbers) was recorded within the wheelpath as compared with that outside the wheelpath. In view of this very slight difference between readings, it can scarcely be regarded as a significant difference in skid resistance other than to suggest the possibility of a trend. On the basis of these readings, any conclusion as to the polishing effect that studs might have on the aggregates or the influence of the aggregates on the skid resistance would be premature.

I believe that we are confronted with a problem that will assume major magnitude in the years ahead. Granted that studded tires prove to be effective as a safety measure, it must be recognized that pavement wear, whether caused exclusively by tire studs or by a combination of studs, salt, and sand, will place new demands on our maintenance and construction efforts and expenditures to remedy or prevent the type of wear that is now occurring.