Control of Pavement Slipperiness

SPECIAL REPORT 101

Asphalt Pavement Cracking

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SPECIAL REPORT 101

L •Control of Pavement Slipperiness•
•Asphalt Pavement Cracking•

Proceedings of the
WESTERN SUMMER MEETING
Cosponsored by the
COLORADO DEPARTMENT OF HIGHWAYS
Held August 12-13, 1968
Denver, Colorado

Subject Area

25 Pavement Design	33 Construction
26 Pavement Performance	35 Mineral Aggregates
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32 Cement and Concrete	51 Highway Safety

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TILTON E. SHELBURNE

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A Tribute

It is altogether fitting to dedicate this Special Report to Tilton E. Shelburne, 65, Director of the Virginia Highway Research Council and a member of the Highway Research Board's policy-making Executive Committee, who died unexpectedly in August.

He had served on some 14 committees of five of the Board's Departments over a 23-year period. He had also received the Board's coveted Roy W. Crum Award in 1956 for distinguished service in the field of highway research.

Mr. Shelburne, who was nationally known for his research work, joined the Virginia Highway Department in 1944 as head of its research program, and in 1948 he became director of the newly-created Research Council, sponsored by the Department and the University of Virginia.

Mr. Shelburne was born in Zionsville, Indiana, on November 19, 1902. He graduated from Purdue University in 1927 and later received his master's degree there. After working for Indiana's Department of Conservation and the State Highway Commission, he returned to Purdue in 1936 as a research engineer and remained there until joining the Virginia Highway Department.

In addition to his research activities for Virginia, Mr. Shelburne was very active in the work of the Highway Research Board. He was chairman of the Department of Design from 1954 to 1963 and a member of the Executive Committee from 1963 until his death. He was also chairman of the National Cooperative Highway Research Program's Design Panel.

At the time of his death he was also a member of the Department of Design, the Department of Economics, Finance and Administration's Committee on Highway Organization and Administration, and Special Committee No. 4 on Conduct of Research.

A Fellow of the American Society of Civil Engineers, Mr. Shelburne was past director of ASCE's District 6 and past president of the Virginia Section. He was also an active member of the American Association of State Highway Officials.

Mr. Shelburne's contributions to highway research brought him many awards. He received the Highway Research Board Award for the best paper given at the Annual Meeting in 1944.

Mr. Shelburne's work in skid prevention also won him wide recognition. He was instrumental in organizing the First International Skid Prevention Conference, held at the University of Virginia in 1958. He was chairman of the executive committee of the American Society for Testing and Materials Committee E-17 on Skid Resistance and past chairman of ASTM's Subcommittee IIa on Field Tests.

The author of numerous technical bulletins and papers, Mr. Shelburne still found time to be active in community affairs. He was a member of the Charlottesville Rotary Club, the Farmington Country Club, and the Westminster Presbyterian Church, in which he was an elder.

Preface

This report contains the technical papers presented at a midyear meeting of the Highway Research Board held August 12-13, 1968, at the Brown Palace Hotel, Denver, Colorado. The objectives of the meeting were to explore specific and timely topics, and to provide opportunity for lower and middle level highway and university personnel from the western and midwestern states to attend and participate in Highway Research Board meetings. This pilot meeting was devoted to the topics of "Control of Pavement Slipperiness" and "Asphalt Pavement Cracking."

The meeting was sponsored by the Highway Research Board and the Colorado Department of Highways.

Contents

PART I—CONTROL OF PAVEMENT SLIPPERINESS

PART II—ASPHALT PAVEMENT CRACKING

Part I
Control of Pavement Slipperiness

A subject of great concern to those responsible for the safety of vehicular traffic is that of highway accidents caused by slippery pavement. It is generally recognized that the problem involves not only the design, construction and maintenance of pavements but also material properties, traffic density and speed, vehicle and tire characteristics, human factors, environmental conditions and many other contributing elements. The program of technical papers and informal presentations at the Denver, Colorado, meeting was designed to focus attention on those aspects of the problem falling under the purview of highway engineers responsible for developing skid resistance requirements and measuring techniques, and programs for the selection of materials, design, construction and maintenance of skid-resistant surfaces. The eleven papers contained in Part I of this report constitute a portion of that program.

The organization of the first part of this report follows generally the organization of the four program sessions. National emphasis on the initiation of programs in skid resistance prompted the inclusion of papers reporting on results of highway skid-resistance inventories using presently accepted testing techniques. For those embarking on statewide inventory programs these papers will provide a working knowledge of sampling techniques and interpretation of test results. Several papers of a basic nature are included, and these will contribute to the knowledge in establishing realistic minimum frictional requirements. Following the first two categories are papers devoted to the design and construction of skid-resistant pavements and to exploring some techniques for restoring or increasing frictional values on existing surfaces. These last papers are aimed at insuring high initial and lasting skid-resistance properties.

Informal presentations and open floor discussions contributing to the general theme of the meeting were not recorded and, therefore, are not included in this report.

A Skid Resistance Study in Four Western States

JOHN A. MILLS, III, Federal Highway Administration, Bureau of Public Roads, Richmond, Virginia

With the increasing emphasis on safety in highway engineering, the Bureau of Public Roads, together with the state highway departments of Colorado, Wyoming, Utah, and New Mexico, conducted a skid resistance study of pavement surfaces in the four states

All testing was conducted with the Bureau of Public Roads skid trailer following ASTM criteria and procedures. A total of 979 tests were conducted on a variety of surface types and highway systems.

Due to its open-textured surface, the bituminous plant-mix seal gave the highest skid-resistance coefficients; followed by chip seals, asphalt concrete and concrete. With chip seals, as with all other surfaces, ADT and age proved to be the most influential factors governing the skid resistance of pavements. A big difference in skid resistance between the inside and outside lanes on four-lane roads was frequently noticed throughout the four states.

It was concluded that the use of bituminous plant-mix seals is the best means now available for insuring a high-quality skid-resistant surface, not only as an overlay on existing surfaces, but also in the construction of new pavements. This study proved beneficial to the Bureau of Public Roads and to the participating four states. A working knowledge of the skid resistance of existing roads is, and will continue to be, extremely helpful to highway engineers.

•THERE has been an increasing awarness of the problem of pavement slipperiness. Together with strength and durability, a good skid-resistant pavement surface has become an integral part of a safe and effective highway system. With the increase in highway traffic speeds and the increase in traffic densities, roads built today must have initial and continuous high skid-resistance qualities.

It is a known fact that skid resistance of pavements is reduced primarily through wear and polish by traffic; however, to what extent the friction characteristics are lessened and the other causes and effects of this reduction are problems the research and highway engineers must face.

Although the automotive and tire industries have contributed a great deal to the research and development of skid-resistant pavements, it is quite obvious that the bulk of improvement will fall upon the highway agencies.

PURPOSE

The purpose of this study was to compare the skid-resistance values of plant-mix seals with those obtained from other conventional surface types, and to show comparisons and trends of skid resistance based on such variables as ADT, age, asphalt content, and type and grade of aggregate used on roads throughout Colorado, Wyoming, Utah and New Mexico.

This study also presents the results of those roadway sections exposed to research construction or maintenance, and of bridge deck testing in the four states.

DESIGN OF STUDY

During the four-week period in June and July 1967, the Bureau of Public Roads' skid trailer measured the skid resistance of roadways preselected by the state highway departments of Colorado, Wyoming, Utah and New Mexico in cooperation with the Bureau of Public Roads' Division offices. The test sites were arranged geographically, with the exception of Colorado, to cover as many different surface types and highway systems as possible. The bulk of testing in Colorado was centered in and around the city of Denver.

The testing equipment consisted of a ¾-ton pickup tow vehicle and a 2-wheeled trailer. The tow vehicle contained a Brush oscillograph for recording the coefficient of friction, a water supply tank and pump, and the brake actuation controls. The trailer included a brush applicator for applying water in front of the test wheel, and instrumented braking system with SR-4 strain gages for the skid resistance measurement, and a standard pavement test tire, E-17.

As outlined in ASTM the trailer was brought to a test speed of 40 mph before the operator actuated the electrical timing equipment. Operating on a 7-sec automatic cycle, a film of water was delivered to the pavement ahead of the test tire and the braking wheel was locked for two seconds (sliding distance of 118 ft). The resulting coefficient of friction based on wheel torque was recorded on a 2-channel oscillograph.

RESULTS

General

During the four weeks of testing, the skid tow vehicle and trailer traveled approximately 4,800 miles throughout the four-state area. A total of 979 tests were taken, but only 800 were used in the correlation of data included in this report. To facilitate the correlation of data, certain assumptions were made.

1. The construction stage of the pavements tested was not considered in the correlation of data.
2. The composition of the previous pavement surface overlaid by a chip seal or plant-mix seal was not considered, as the coefficient of friction measured was only on the existing surface course.
3. The primary objective of this study was to compare the skid resistance of various pavement surfaces and not to study the effect of speed, therefore, all data used in the correlation and comparisons were taken from the standard test established by ASTM (40-mph speed of trailer).
4. The tests run on paint stripes, between wheel tracks and on shoulders are not included in the correlation of the 800 tests.
5. Those tests run on slick spots or other sections that are not truly characteristic of the entire roadway were not included in the correlation.

Table 1 gives a summary of coefficients by lanes by states; see also Figures 1 through 5.

Wyoming

Plant-Mix Seals—The Wyoming plant-mix seals gave the highest friction values recorded in the four states A close analysis of the materials and quality control used on the plant-mix seals indicates that, due to the many variables involved in and on the pavement, only the following general trends can be concluded:

1. Bank gravel gave higher results than limestone, but this was expected as the limestones generally tend to polish more readily with age and wear.
2. The greater the percentage of fractured faces in the material retained on the No. 4 screen, the higher the coefficient of friction. (The author feels 100 percent fractured faces is most desirable for a high-quality, skid-resistant plant-mix seal).
3. The percent of asphalt varied from 5. 2 to 7.4 with the majority of test sections containing between 6 and 7 percent. The trend in plant-mix seals is that the coefficient of friction increases as the percent of asphalt content decreases; however, it should be

TABLE 1

SUMMARY OF COEFFICIENTS BY LANES BY STATES

State	2 Lanes—1 Direction[a] Driving	2 Lanes—1 Direction[a] Passing	1 Lane—1 Direction[b]
		Colorado[c]	
Plant-mix seals	7 02/15 = 0 47	0 57/1 = 0 57	2 86/7 = 0 41
Concrete	9 62/26 = 0 37	0 90/2 = 0 45	None Tested
Asphaltic concrete	20 40/63 = 0 32	0 84/2 = 0 42	5 89/14 = 0 42
Chip seals	None Tested	None Tested	None Tested
		Utah	
Plant-mix seals	12 00/25 = 0 48	9 04/17 = 0 53	22 64/43 = 0 53
Concrete	3 43/8 = 0 43	4 92/11 = 0 45	None Tested
Asphaltic concrete	9 45/22 = 0 43	5 07/12 = 0 42	2. 55/5 = 0 51
Chip seals	6 06/12 = 0 51	4 57/8 = 0 57	6 07/11 = 0 55
Road mix	None Tested	None Tested	2 21/8 = 0 28
		Wyoming	
Plant-mix seals	6 63/12 = 0 55	2 97/5 = 0 59	16 57/31 = 0 53
Concrete	3 63/9 = 0 40	1 59/3 = 0 53	None Tested
Asphaltic concrete	9 78/21 = 0 47	6 78/12 = 0 57	4 37/9 = 0 49
Chip seals	7 26/17 = 0 43	4 27/7 = 0 61	6 90/14 = 0 49
		New Mexico	
Plant-mix seals	16 86/34 = 0 50	12 75/23 = 0 55	None Tested
Concrete	11 83/26 = 0 46	14 01/29 = 0 48	None Tested
Asphaltic concrete	29 21/62 = 0 47	27 26/50 = 0 55	26 16/54 = 0 48
Chip seals	None Tested	None Tested	8 13/18 = 0 45

[a] In the case of 3 lanes in 1 direction, the middle and inside lanes are in the passing lane category for 2 lanes—1 direction, and the right lane is in the driving lane category for 2 lanes—1 direction

[b] In the case of a roadway having 1 lane in 1 direction with a climbing lane, the test results for each lane are shown as 1 lane—1 direction Ramps and deceleration lanes are classified with 1 lane—1 direction

[c] The tests on plant-mix sand seals in Colorado were included in the category of asphaltic concrete

noted that all sections tested gave high values. Further, while this trend is noted from a skid resistance standpoint, other factors should be considered before attempting to significantly reduce the percent of asphalt.

Portland Cement Concrete—Due to the small number of rigid pavement sections, only 12 tests were conducted with an average friction coefficient of 0.44. This value is only 0.03 less than the maximum value attained on rigid pavements in the four states, and no significant characteristic was outstanding. As in flexible pavements, bank gravel aggregates gave higher values than did limestone. One test was taken on the tunnel floor paving along I-80. The Class B concrete surface yielded a value of 0.35, which is 0.09 lower than the rigid pavement average for Wyoming and 0.06 lower than the average for concrete structures in the four states.

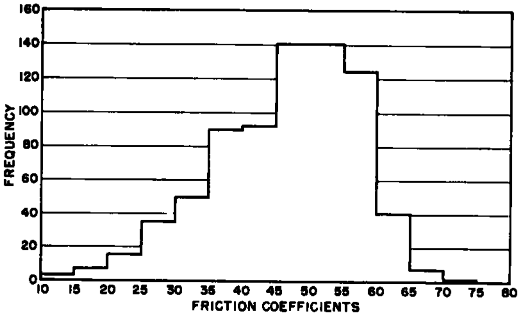

Figure 1. All surface types.

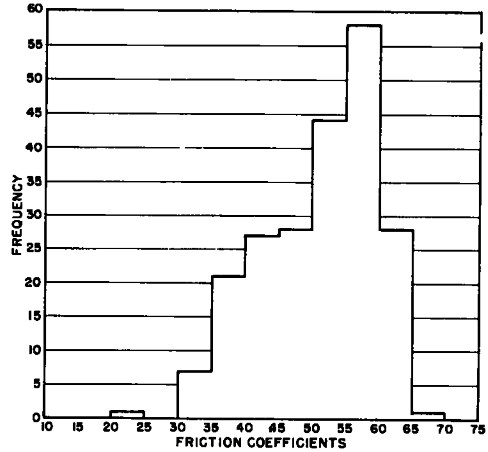

Figure 2. Plant-mix seal.

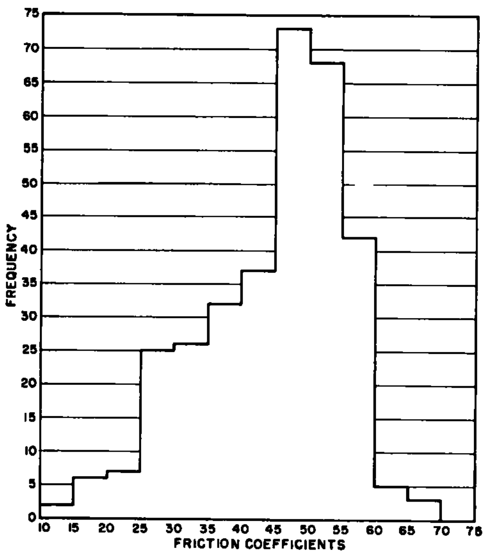

Figure 3. Asphalt concrete.

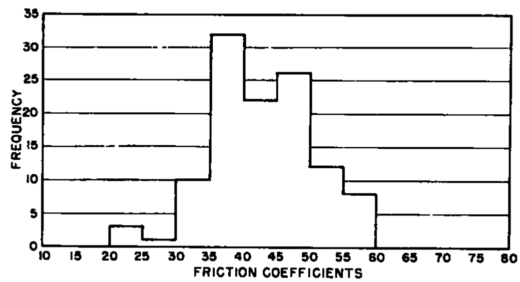

Figure 4. Concrete.

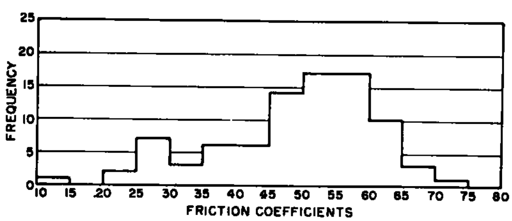

Figure 5. Bituminous-chip seal.

Asphaltic Concrete—All tests on plant-mix asphaltic-concrete sections in Wyoming were along Interstate routes, and a comparison of the average values indicates a definite trend in the coefficient of friction obtained in the outside or traveled lane and the inside or passing lane. The outside lane constantly indicated a lower coefficient than the inside lane. This is not surprising as the outside lane is exposed to a much higher traffic density than the inside lane. This trend points out a need for anticipating this decrease in skid resistance and compensating for it either during initial construction or through maintenance operations.

Chip Seals—To substantiate further the above statement regarding lane differences, the average values for chip seals by lanes show a difference of 0.18. This difference between lanes is the largest in comparison with any pavement type in all four states. The sections tested were composed of limestone, quartzite and granite, and scoria (burned shale), with the majority of surfaces having limestone chips. A comparison

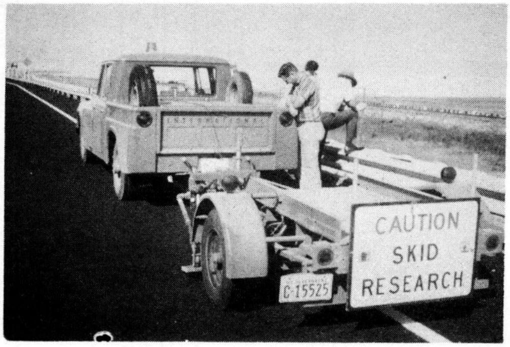

Figure 6. Tow vehicle and skid trailer.

of the native material shows granite and quartzite having a skid resistance value of 0.45, limestone having a value of 0.50 and scoria chips having a coefficient of 0.63.

One section of highway in northwest Wyoming was tested where "deslicking operations" of an old limestone chip seal were in progress. An average of the 15 tests on the old surface gave a coefficient of friction of 0.16. ["The coefficient of friction at incipient skid on untreated ice varied from 0.08 to 0.20 (1)."] An average of the tests run on the deslicked area gave a coefficient of 0.60. This increase from 0.16 to 0.60 is on a section one month old, and the answer to how long the deslicked surface will maintain this high coefficient remains to be determined. To accomplish the deslicking a repaver heated the old surface, and sand and gravel were spread and rolled on the roadway.

Colorado

The majority of tests representing Colorado data were run in or near the city of Denver. Table 2 indicates that the values obtained are lower on an average than values for similar pavement types tested in the other states. It is possible that the heavy ADT in and near Denver could be a factor contributing to the lower values.

Plant-Mix Seals—All plant-mix seal sections tested were composed of granite; consequently, a comparison of aggregate type relative to friction coefficients is impossible.

The composition of the plant-mix seals tested can be separated into two groups—a clear crack granite aggregate with 100 percent fractured faces on the plus 4 material, and an asphalt content of 7 percent; and a clear crack granite aggregate 85 percent with fractured faces and an asphalt content of 8 percent. A comparison of the average values obtained on the two compositions shows a difference of better than 0.10, with the 7 percent asphalt and 100 percent fractured faces composition being higher. All sections tested were within several months of being the same age, and it is believed that the ADT difference on the two compositions is the primary reason for the difference in friction coefficient. With only one test run in the inside lane, a valid comparison of lanes for this pavement is impossible.

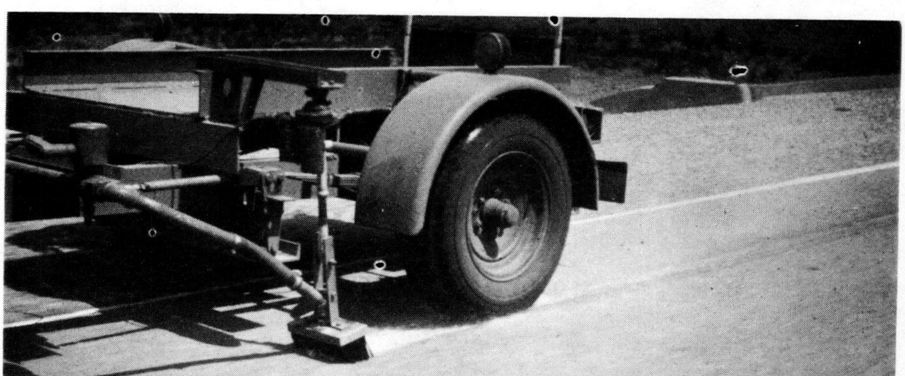

Figure 7. Skid trailer in braking cycle.

TABLE 2

SUMMARY OF PAVEMENT COEFFICIENTS BY STATE

Pavement Type	Colorado	Utah	Wyoming	New Mexico	4-State Average
Plant-mix seals	$\frac{10.45}{23} = 0.45$	$\frac{43.68}{85} = 0.51$	$\frac{26.17}{48} = 0.55$	$\frac{29.61}{57} = 0.52$	$\frac{108.92}{211} = 0.52$
Concrete	$\frac{10.52}{28} = 0.38$	$\frac{8.34}{19} = 0.44$	$\frac{5.22}{12} = 0.44$	$\frac{25.84}{55} = 0.47$	$\frac{49.93}{11} = 0.44$
Asphaltic concrete	$\frac{27.13}{79} = 0.34$	$\frac{17.07}{39} = 0.44$	$\frac{20.93}{42} = 0.50$	$\frac{82.63}{166} = 0.50$	$\frac{147.76}{326} = 0.45$
Bit-chip seals	None Tested	$\frac{16.70}{31} = 0.54$	$\frac{18.43}{38} = 0.49$	$\frac{8.13}{18} = 0.45$	$\frac{43.26}{87} = 0.50$
Road mix		$\frac{2.21}{8} = 0.28$	None Tested	None Tested	$\frac{2.21}{8} = 0.28$

In the above table the figures in the denominators denote the number of tests on the respective pavements.

Portland Cement Concrete—The discussion of the rigid pavement results is restricted, since only limited information on pavement compositions was available. An analysis of the test data does not show any definite trends. In some cases, the values obtained on sections 12 and 15 years old were higher than those only 2 or 3 years old. As with other pavements, there is an indication that friction values decrease faster on those sections exposed to higher ADT.

A section of longitudinally grooved rigid pavement was tested near Peckham. The section was subjected to intermittent grooving to decrease surface roughness, and the results show an increase in friction coefficient from 0.35 on the ungrooved section to 0.40 on the grooved section. These tests were run in the outside lane, with the inside lane yielding a value of 0.48. Further tests were run on the same pavement six miles from the grooved area, and the results show a coefficient of 0.40 (ungrooved). The data, coupled with the information in the discussion of Utah's results on grooved pavements, lead to the conclusion that grooving, although highly effective in reducing "hydroplaning," does little to increase skid resistance.

Asphaltic Concrete—Seventy-nine tests were averaged in determining the skid resistance value for Colorado. The value of 0.34 is the lowest for any surface type in any state, and this low value is believed to be due to the wide use in the Denver area

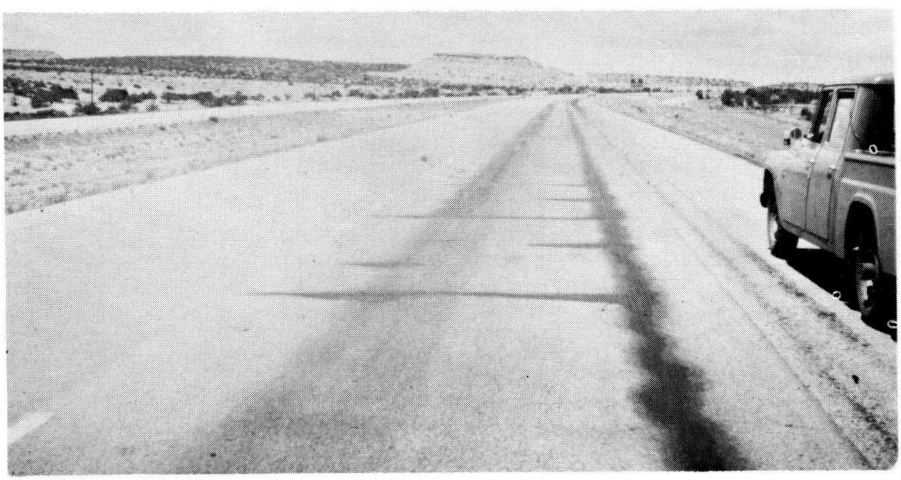

Figure 8. A plant-mix seal surface with flushing in the outside lane, outside lane coefficient = 0.38 and inside lane coefficient = 0.58, age = $1\frac{1}{2}$ yr, ADT = 6590, limestone, 75 percent fractured faces, 6.5 percent asphalt content.

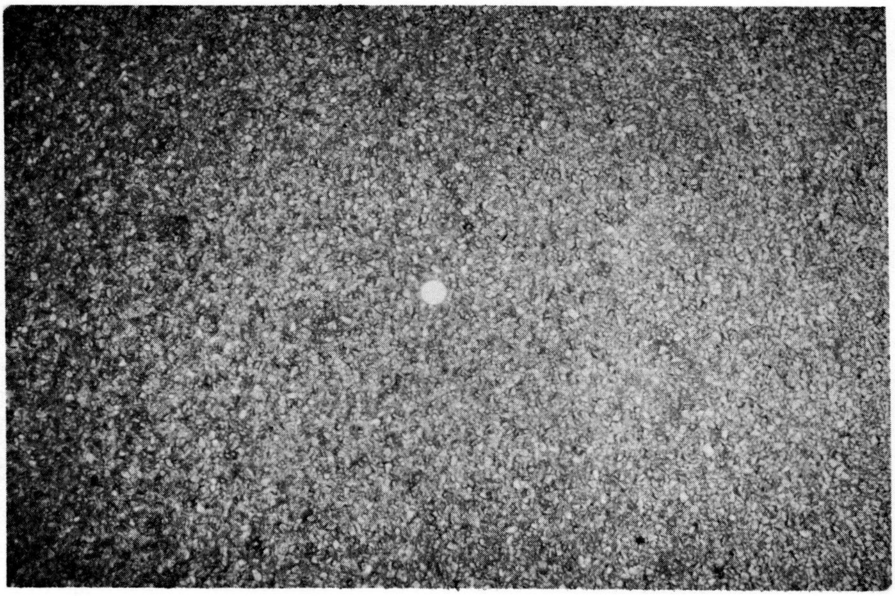

Figure 9. A close-up of the inside lane in Figure 8, where coefficient = 0.58.

of different asphaltic seals and overlays. Coated aggregate on fog-sealed mat, flushed surfaces, fine-grain sealed mats, and dense-graded plant-mix were the three more prominent surfaces tested. Since only two tests were run on the inside lane of a 2-lane, 1-direction highway, a lane comparison for this surface type was not considered. There were no chip-sealed surfaces tested in Colorado for use in this study.

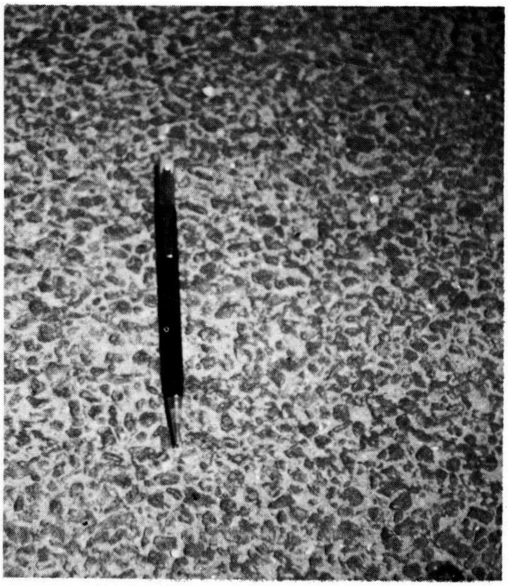

Figure 10. A plant-mix seal surface with coefficient = 0.62, age = 2 yr, ADT = 95, bank gravel, 95 percent fractured faces, 6.5 percent asphalt content.

New Mexico

Plant-Mix Seals—Fifty-seven tests were conducted on plant-mix sealed projects with an average value of 0.52 (the average for all the plant-mix seals tested in the four states). The testing is more evenly divided between the inside and outside lanes than in the other States, making the comparisons between lanes more substantial.

Of the nine projects with plant-mix seals, eight were tested in both the inside and the outside lanes with an average difference of only 0.05. This small difference could result from the greater percentage of vehicles driving in the inside lane than in other states, or perhaps the plant-mix seals in the outside lane are holding up extremely well under traffic. The aggregate compositions used in the plant-mix seals were sand and gravel, limestone, and gravel. A comparison of the friction coefficients by aggregate type shows that both sand and gravel, and gravel yielded a value of 0.56; limestone is 0.09 lower with a value of 0.47. This difference is believed to be caused by polishing of the limestone.

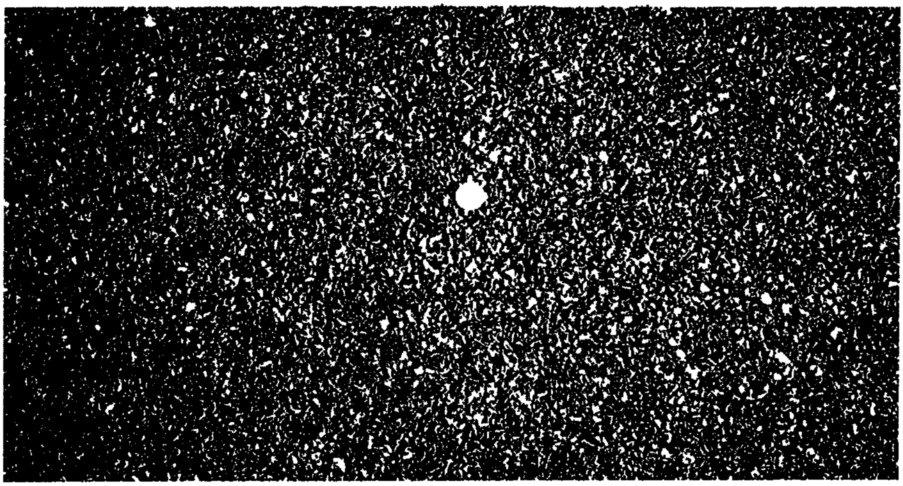

Figure 11. A plant-mix seal surface with coefficient = 0.38, age = 2 yr, ADT = 2880, limestone, 75 percent fractured faces, 6.5 percent asphalt content.

Since all plant-mix sections tested had 75 percent fractured faces on the plus 4 material, a comparison of this property is omitted. The percent of asphalt content ranged from 6 to 7 percent, with no definite trend as to the asphalt content influencing the coefficient of friction. A research project consisting of a plant-mix seal with a rubber additive was tested on a Federal-aid primary route in Santa Fe. The additive, Goodyear Plyopave LO-170, was 3 percent by weight of the mix and yielded coefficients of approximately 0.02 below the state average for the driving and passing lanes. Even though the coefficient of friction was not appreciably affected by this additive, it may extend the service life of the mix through increased flexibility and decreased hardening of the asphalt.

Portland Cement Concrete—All concrete sections tested were on Interstate routes, with an ADT range from 2,000 to 5,000. A graphical study of the data based on age with the previously mentioned ADT range disclosed a very gradual decrease in the friction coefficient after the second year. For the first two years the pavement surface seemed to experience fluctuations, which could have been caused by different types of surface finishes or the wearing down of the mortar. New Mexico had the highest average for rigid pavements in the four states (Table 2). The aggregates used were gravel, limestone, basalt, and sand and gravel, with limestone being the most predominantly used and yielding the highest coefficient average. A comparison showed the following averages: limestone 0.51, basalt 0.47, gravel 0.41 and sand and grave 0.39. The aggregates other than limestone received only limited testing. The limestone sections tested ranged in age from 1½ to 7 years and the percent of wear by the abrasion tests ranged from 19.6 to 30 percent.

Figure 12. Good surface drainage from a plant-mix seal.

Asphaltic Concrete—Due to limited information, a comparison of the percent of

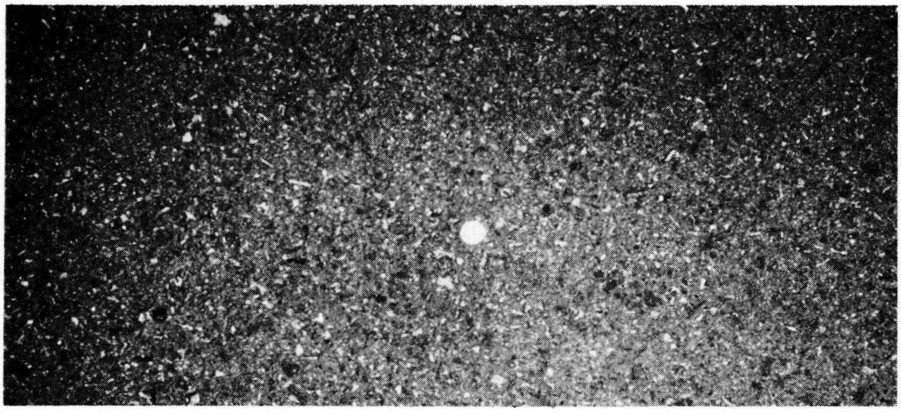

Figure 13. An asphaltic-concrete surface, coefficient = 0.53, age = 1½ yr, ADT = 945, limestone, 60 percent fractured faces, 6.0 percent asphalt content.

Figure 14. An asphaltic-concrete surface with flushing in the outside lane, outside lane coefficient = 0.39 and inside lane coefficient = 0.55, age = 3 yr, ADT = 6210, basalt, 60 percent fractured faces, 5.75 percent asphalt content.

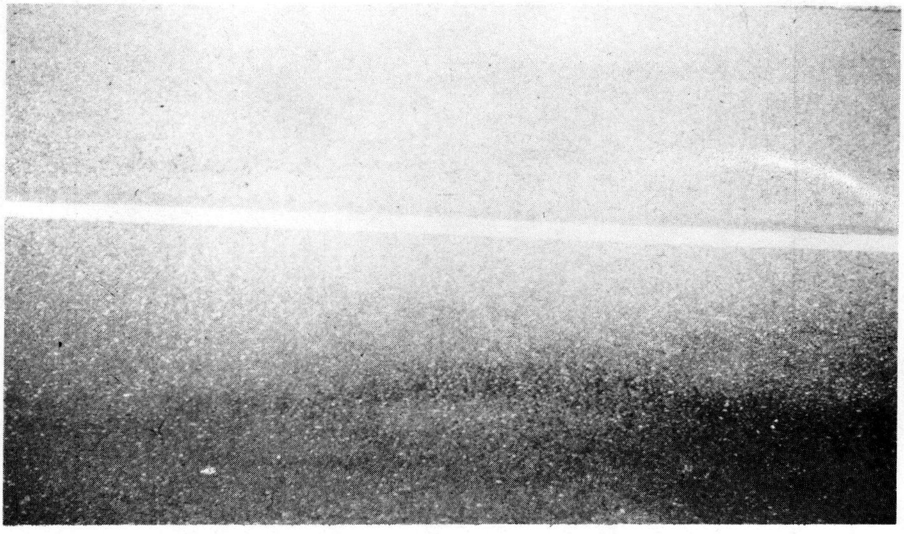

Figure 15. Close-up of Figure 14.

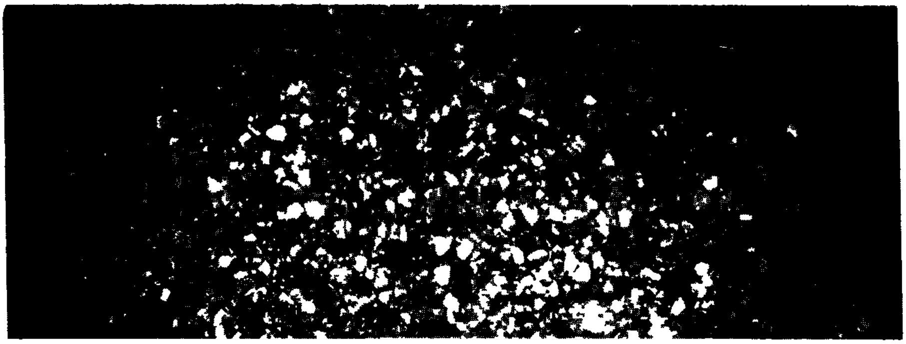

Figure 16. A chip seal surface with coefficient = 0.63, age = 3 yr, ADT = 73, quartzite and sandstone.

fractured faces of the plus 4 material in the asphaltic-concrete sections was not made. Of the 166 test results used in the analysis of asphaltic concrete, there were four different aggregate types tested and a comparison of the four showed a difference of only 0.04 between the lowest, basalt, and the highest, gravel (gravel 0.51, sand and gravel 0.50, limestone 0.49, and basalt 0.47). Even though the majority of tests were run on Interstate routes, the one-way ADT ranged from 125 to 5,465 with the average for the state approximately 1,500. A research project was conducted on an asphaltic-concrete surface treated with different types of fog seals. The following averages are based on three to eight tests on each section: Reclamite 0.49, Gilsonite 0.43, MC70 0.35, SC70 0.52, SSKH 0.40. These fog seals are being studied for surface rejuvenation.

Chip Seals—Only 18 tests were used for an average on chip seals, and due to the limited information available, very little could be gained from the sections tested. The only aggregate used was gravel and the test sections, which ranged from one to six years, showed no significant trends.

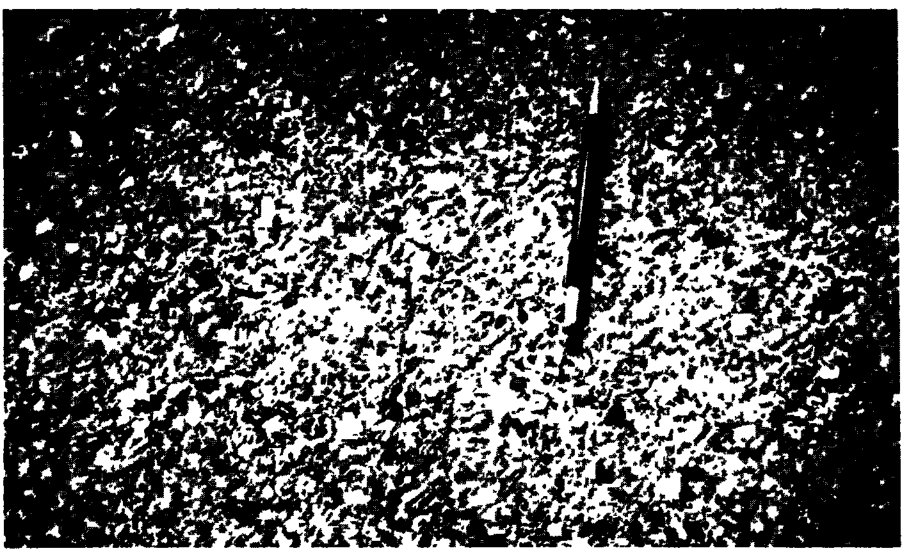

Figure 17. A chip seal with a coefficient = 0.60, age = 4 yr, ADT = 965, scoria (burned shale).

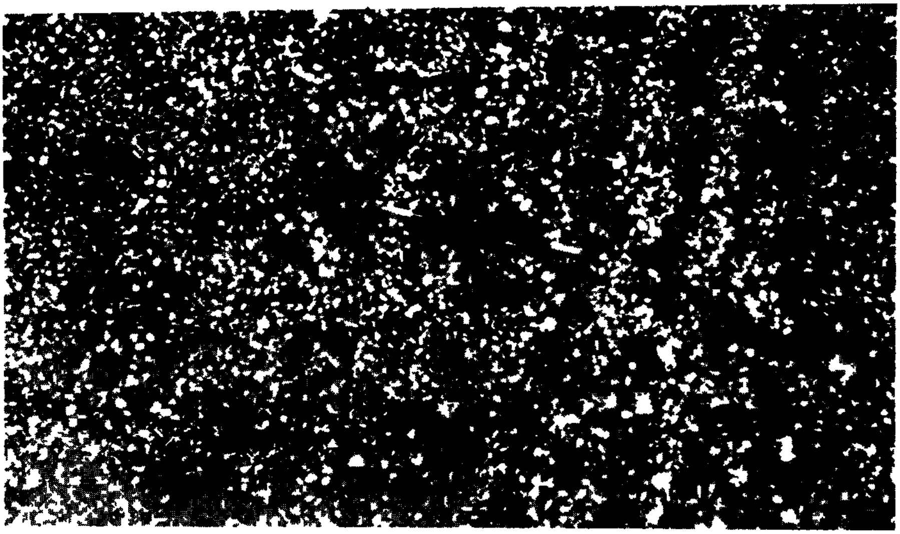

Figure 18. A chip seal surface before "deslicking operations" where coefficient = 0.16, ADT = 355 (other information not available).

Utah

Plant-Mix Seals—More plant-mix seals were tested in Utah than in any of the other states with an average value of 0.51. Over half of the tests were conducted on 1-lane, 1-direction roads, and these tests had the same average as the passing lane of the 2-lane, 1-direction roads. The only obvious reason for this similarity is that a great majority of the testing on 1-lane, 1-direction routes was on Federal-aid primary routes with reduced ADT.

The aggregate combinations used in the plant-mix seals were limestone, quartzite, sandstone, and basalt. One section contained open hearth slag. Due to the limited information available, a valid comparison as to percent of fractured faces, aggregate

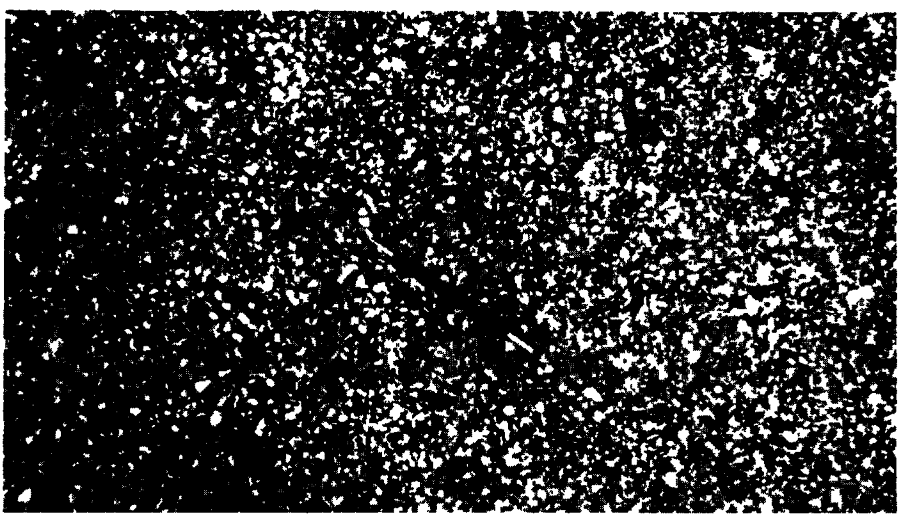

Figure 19. The same chip seal surface shown in Figure 18 after "deslicking operations" where coefficient = 0.60.

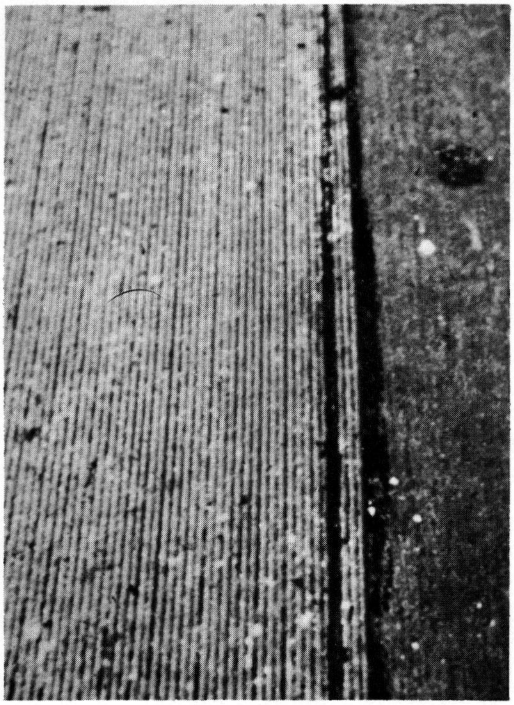

Figure 20. A concrete surface with longitudinal grooving on the left and the original surface on the right; coefficient on grooved section (left) = 0.40 and coefficient on ungrooved section (right) = 0.35, age = 15 yr, ADT = 2850 (other information not available).

used, and percent of asphalt was not made. The data available indicated a desirable minimum of 74 percent fractured faces with an asphalt content range between 6 and 7 percent.

One section was tested, which had been exposed to deslicking operations. The operation consisted of heating the pavement surface without applying any cover material. Six tests were run on this section, and no conclusions were made. Two tests on sections adjacent to the deslicked area gave an average of 0.56. Two tests on bleeding sections adjacent to the deslicked area gave an average of 0.42, and the two tests on deslicked sections gave values of 0.33 and 0.59.

Portland Cement Concrete—All testing of concrete was on Interstate routes with only 19 tests used for averaging. A comparison of values obtained on the driving and passing lanes showed a difference of only 0.02. This small difference between lanes is believed to be due to uniform wear of the concrete surface. There is a definite trend for the friction coefficient to decrease with age, which has been the case for most of the other surface types. Aggregate type was not available on concrete so a discussion of this property will be omitted. The test results of a concrete bridge deck exposed to grooving will be discussed in the section on bridges.

Asphaltic Concrete—The most widely used aggregates in asphaltic concrete were limestone, sandstone and quartzite, and limestone and quartzite. A comparison of the aggregate types showed quartzite and limestone having the highest value, 0.46, with sandstone and quartzite having 0.44, and limestone having the lowest, 0.35.

An irregularity was noticed in the lane comparison of asphaltic concrete between the driving and the passing lanes. It is believed that this decrease in the passing lane value is due to unbalanced field testing.

Chip Seals—The chip seals in Utah yielded the highest friction coefficient of all the states. The aggregate chips tested were combinations of limestone, sandstone, and quartzite. One section of Interstate tested had a blended aggregate composition of 60 percent mineral aggregate and 40 percent slag. In comparing the results of the different aggregate compositions, the reader should bear in mind that in several cases the averages are based on results obtained on only one project. Combinations of both sandstone and quartzite, and blended mineral aggregate and slag gave the highest coefficient of 0.62; quartzite and limestone had 0.57; quartzite, 0.51; and limestone, sandstone, and quartzite had the lowest, 0.47. Correlation of the chip seals data indicated more of a coefficient decrease with ADT than with age.

Road Mix—The only road mix section tested was in Utah. Several tests were conducted on the shoulder, between the wheelpaths, and on the centerline of the roadway, and those tests averaged together were only 0.03 higher than the average of tests conducted in the wheelpath. This is an indication of the small effect the ADT of 330 vehicles over a 2-year period had on this road-mix surface. The road mix was composed of a limestone aggregate with 90 percent fractured faces and a ± 4.3 percent asphalt content.

TABLE 3

SUMMARY OF BRIDGE DECK COEFFICIENTS OF THE FOUR-STATE AREA

Pavement Type	2 Lanes—1 Direction Driving	2 Lanes—1 Direction Passing	4-State Average
Concrete	$\frac{10\ 45}{25} = 0\ 42$	$\frac{2\ 79}{7} = 0\ 40$	$\frac{13\ 24}{32} = 0\ 41$
Plant-mix seals	$\frac{1\ 03}{4} = 0\ 52$	None Tested	$\frac{1\ 03}{2} = 0\ 52$
Bituminous Miscellaneous seals[a]	$\frac{4\ 55}{12} = 0\ 38$	None Tested	$\frac{4\ 55}{12} = 0\ 38$

[a]Includes 10 tests on a sand seal overlay, 1 plant-mix overlay, and 1 Jennite sealer

Bridges

Forty-six tests were run on bridge decks in the four state area and, with only one exception, the testing was done at random. That one exception was a concrete structure in Salt Lake City, where the state highway department and an equipment manufacturer were engaged in a research project (Table 3).

Most of the testing was conducted in Colorado, since structures comprised much of the Interstate routes tested. The concrete and bituminous miscellaneous seals in Colorado yielded slightly higher values on bridge decks than they did in the driving lane. It is believed that the differences of 0.05 and 0.06 are primarily due to the workmanship of the surfaces during construction. In Wyoming, two plant-mix seal decks were tested, and the results differed by only 0.03.

A comparison between the driving and passing lanes for concrete shows the passing lane value 0.02 lower than the driving lane. This irregularity is due to the test results on the previously mentioned research project in Utah, where three low values were taken on grooved sections. Reducing the possibility of hydroplaning and reducing the roughness were the primary reasons for the grooving. At the present time equipment manufacturers are preparing equipment which will increase the friction coefficient as well as reduce hydroplaning and roughness. The testing of the different surface types was not balanced for a conclusive comparison.

CONCLUSIONS

This study indicates the increased attention to the subject of skid resistance and the concern of the Bureau of Public Roads in (a) development and use of equipment for measuring skid-resistant qualities of pavement surfaces, (b) development of materials and construction techniques for providing high skid-resistant qualities on new pavements and maintaining them after the pavement is exposed to traffic, and (c) establishment of programs for measuring and analyzing the skid-resistant qualities obtained on conventional types of surfaces now being constructed.

The rugged trip covering approximately 5,000 miles demonstrated the mobility, dependability, and expeditious manner of testing with the locked-wheel skid trailer.

With the many variables present for each pavement, it was extremely difficult to pinpoint an exact cause or reason for a pavement's behavior. Only through comparison of pavements exposed to similar conditions over a period of time can any trends or results be obtained. It was for this reason that ADT, age, aggregate type, percent of fractured faces, and asphalt content were the main properties selected for consideration and comparison.

Table 2 shows the friction coefficients by state and the four state average. A comparison shows plant-mix seals have a coefficient of friction higher than the other conventional surface types. The main reason for this is its open-textured surface, which provides for better drainage of water. A trend observed on plant-mix seals during the skid study was an increase in skid resistance with aging of the pavement. This increase is due to traffic "wearing off" the asphalt coating of the aggregates. However, the oldest plant-mix seal tested was five years old, and further study is needed on this pavement type. Eager (2) states that typical specifications for plant-mix seals include the following requirements:

Gradation of aggregate:

Pass ½-in sieve	100%
Pass ⅜-in sieve	95-100
Pass No. 4 sieve	30-50
Pass No. 8 sieve	15-30
Pass No. 40 sieve	0-10
Pass No. 200 sieve	0-5

* * *

Los Angeles abrasion	40 max.
Fractured faces	75% plus on +No. 4 size
Soundness	12% max.
Retained coating	75%+ (by AASHO T182 or other suitable stripping resistance test. May be necessary to use an additive to obtain adequate stripping resistance. Hydrated lime or chemical additives are frequently used)
Mixing temperature	275 F max (less if possible)
Placing temperature	225 F min
Grade of asphalt	60-70 or 85-100
Percentage of asphalt	6-8 (use as high as possible)
Rate of placing	⅝ in - ⅜ in compacted
Tack coat	Not always used but if so at 0.05-0.10 gal per sq yd using a light RC or emulsion.

The percentages of fractured faces on the No. 4 material were investigated quite extensively in the preparation of this paper, and it is recommended that 100 percent fractured faces be included in design criteria.

The cost of plant-mix seals was not discussed; however, based on figures from the Bureau of Public Roads, "the cost of a chip seal would probably be less than one-half the cost of a plant mix seal on a square yard basis. However, the thickness would only be about one-third so that the cost per inch of depth is actually less for the plant mix seal (2)."

The friction coefficients for concrete and asphaltic concrete are practically the same for the four state averages and are closely related within each state. A great deal of aggregate polishing was experienced in both pavement surface types; however, it is believed that aggregate polishing was much more prevalent in the asphaltic concrete than in the concrete. An overall comparison of the aggregate types used showed limestone polishing more readily and giving lower values. Gravel and granite, and quartzite gave consistently higher results than did other aggregates.

Concerning concrete surfaces, even though they rank next to last in average friction coefficient, the decrease rate of skid resistance with age is less for this type of pavement than any other type tested.

The use of sand seal overlays indicated a decrease in the skid resistance in cases where high ADT was experienced. Those asphaltic surfaces where chips and other fine aggregates were removed due to wear and age retained the surface water and gave low skid values. Other fog and flush seals used for map protection and rejuvenation gave both high and low values.

The chip seal surfaces gave the second highest average skid value; however, the reader should keep the following in mind when analyzing this study: (a) there were fewer tests run on this surface type than on plant-mix seals, asphaltic concrete, or concrete; (b) the test sections consisted primarily of primary and secondary projects; and (c) the ADT in most cases was relatively low.

Chip sealed surfaces showed the effects of traffic more than the other surfaces as there was a more pronounced decrease in skid resistance with the increases in ADT. There were no definite conclusions as to aggregate preference for chip seals, as all types gave favorable results. Open hearth slag and scoria chips yielded extremely high skid values.

With chip seals, as with all other surfaces, ADT and age proved to be the most influential factors governing the skid resistance of pavements. Good workmanship and other construction procedures are most important in the preparation of a good skid-resistant surface.

The big difference in friction coefficients between the driving and passing lanes could be a serious problem. A need for further study and research into this problem is definitely needed.

Concrete bridge decks proved to have a lower skid resistance than the decks which had been overlaid with bituminous mixes. More testing of other deck surfaces should be performed before definite conclusions are drawn, as only 42 tests were included in this study.

The histograms show the frequency of occurence of the friction coefficients for the different surface types.

RECOMMENDATIONS

This study proved beneficial not only to the Bureau of Public Roads, but also to the four participating states. A working knowledge of the skid resistance of existing roads is, and will continue to be, extremely helpful to highway engineers. It is recommended that all states incorporate into their systems a surface condition study of skid resistance to be conducted periodically. From the standpoint of safety, skid resistance is becoming a more important factor in developing safer highways, and a good skid resistance program is a giant step towards achieving this goal.

Based on the results and analysis of this study the author recommends the use of plant-mix seals as the best means now available for insuring a high quality skid-resistance surface not only as an overlay on existing surfaces but also in the construction of new pavements.

ACKNOWLEDGMENTS

The author wishes to acknowledge the cooperation of the U. S. Bureau of Public Roads and the state highway departments of Colorado, Wyoming, Utah, and New Mexico. The opinions and conclusions expressed are those of the author and not necessarily those of the Bureau of Public Roads. Special appreciation is expressed to Emery L. Shaw and William L. Eager for their advice and guidance throughout the study and the preparation of this paper.

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Development and Results of a Skid Research And Road Inventory Program in Pennsylvania

LEO D. SANDVIG, LOUIS M. MACGREGOR, and RICHARD K. SHAFFER,
Pennsylvania Department of Highways

The Pennsylvania Department of Highways initiated a statewide annual skid survey in 1963 using the single-wheel skid trailer developed at Pennsylvania State University. Five of these surveys have served to guide the maintenance bureau in providing safer highways, and have contributed to the overall direction of the research program. Data collection methods and testing techniques were perfected during this time and carried over to a more strenuous research program currently in progress. The scope of past and current research, both in-house and department-sponsored, is described briefly.

Results of the five completed surveys have been analyzed. Despite the geographical, hence the aggregate, diversity of statewide data, some relationships formerly only suspected are now apparently established. These results are presented in this paper.

Expansion of Pennsylvania's road inventory and research programs is imminent. New, fully automated skid testers are being acquired; one or two may be assigned to critically located regional offices to maintain closer data control and provide better service. All data will be routed through the central computer for analysis providing the combined service-research benefits originally intended. The Department will implement the research by issuing specifications providing for skid-resistant surfaces.

•ALTHOUGH the skidding problem, in one form or another, has prevailed for many years, its threat to human lives was not seriously considered until the advent of the automobile. Concurrent with its development, miles of hard-surfaced, high-speed highways were presented to the motorist for his pleasure and convenience. Although most of these roads have been improved or replaced to provide safer, faster highways in the form of divided and limited-access roads, many of the old roads still remain. Despite all the safety features built into present highways, more people are being killed annually than ever before. In Pennsylvania alone, 2131 traffic fatalities were recorded in 1967 while on the national scene, over 53,000 deaths were reported. The reasons are clear: more people are driving more cars more miles faster. The part that skidding plays in the latter observation is well documented in the literature (1, 2). The seriousness of the problem is emphasized, however, by the activities of private and public institutions in their efforts to resolve it.

INITIAL RESEARCH

The Pennsylvania Department of Highways was among the first to dedicate extensive funds to skid resistance research. In 1960, Pennsylvania State University was engaged to conduct a pilot program in one of the state's engineering districts that historically has had skidding problems (3).

The objectives of the testing program were to determine the degree of slipperiness as a guide for assigning priorities for resurfacing, and to gain insight into the causes of slipperiness. These objectives were met by utilizing the prototype single-wheel skid trailer designed and built by Penn State and the assistance of personnel from the Highway Department's Bureau of Materials, Testing and Research. Nineteen sites in District 10 were selected for testing. Eleven of these areas exhibited Skid Numbers well below 35 and were classified as slippery. Close analysis of these sites revealed the absence of sand-size particles in the polished pavement surface. Chemical analysis of the limestone from one of the slippery surfaces yielded less than an acceptable minimum fraction of silica sand required for adequate skid resistance. Surfaces exhibiting the greatest skid resistance showed sand-sized particles in the aggregate and a sandpaper-like appearance of the surface. The polishing effect of traffic was established by comparing the skid numbers in the wheel tracks with those in the center strips. From the results of these tests, recommendations were formulated for long- and short-range programs to improve the skid resistance of Commonwealth roads.

So encouraging was this experiment that a second one was planned for 1962. This time, the entire state was included for the purpose of establishing the suitability of the Penn State tester design for routine surveys and of initiating a continuing program of annual surveys of the highway system (4). The test schedule, prepared by the Department and covering a six-week period, included all 11 districts and the Pennsylvania Turnpike. The survey sites, selected by district personnel for known and suspected slipperiness, averaged about 23 projects per district. The skid resistance data showed that each district had its share of slippery pavements. The mean for all projects, excluding the Turnpike, was 34.4 at 35 mph while the equivalent figures for the Turnpike were 44.1 at 35 mph and 30.2 at 65 mph. The main surface types appeared not to affect the skid resistance significantly, whereas the aggregate had a pronounced effect on the readings obtained. Gravel surfaces exhibited the highest skid resistance whereas limestone pavements produced the lowest readings. Skid resistance, in general, was shown to decay with pavement age, rapidly at first and more gradually later on. The number of passes on the pavement surface, as computed from the average daily traffic (ADT), was actually found to be a more sensitive indicator of the decay of skid resistance than age alone.

THE JOINT ROAD FRICTION PROGRAM

In 1961, the Department initiated a cooperative research study known as the Joint Road Friction Program with Pennsylvania State University. The objectives of this ongoing study are to provide the Department with skid resistance measuring equipment and to conduct research into the fundamentals of skid resistance with the development of a scientific basis for controlling it. Since 1963, the Bureau of Public Roads has been a participating sponsor in this research.

An early report (5) deals with the development, selection, and performance of the single-wheel skid trailer. Whether for research purposes or for routine skid surveys, the following ten requirements are fundamental to the design of a skid trailer:

1. Meaningful measurements,
2. Precision of test data,
3. Minimum data processing,
4. Balanced coverage and test cycle frequency,
5. Adequate range of operation,
6. High degree of mobility and maneuverability,
7. Minimum traffic interference,
8. Ruggedness,
9. Economy of operation, and
10. Comfort and safety of crew.

The ability of a two-wheel trailer to measure skid resistance in either the left or right wheel track is of advantage in specialized experiments, but it is of little value for highway surveys or most research. Tests have shown that the Skid Number is the same in both wheel tracks of no-passing lanes, whereas the left wheel track exhibits lower readings where passing is permitted. Since highway engineers are generally interested in the lowest skid resistance of a section of pavement, a single-wheel trailer, so mounted that it normally runs in the left wheel track, is perfectly adequate. The pneumatic loading system of the single-wheel design permits changing the wheel load during operation and more importantly, it permits automatic and close control of the wheel load to a predetermined value. The pneumatic loading system also allows for raising the trailer wheel off the road for transportation between test sites. This is a definite advantage in some of the mountainous terrain in Pennsylvania.

In the course of research into the causes of slipperiness, various laboratory and field test equipment has been developed and constructed to measure specific factors which have a bearing on skid resistance. While some of this equipment has been described in various technical and research publications, the information is too scattered and incomplete to serve as a useful reference for workers in this field. One phase of this research program is embodied in a report which defines the test function, operating principle, specifications, and test procedure for laboratory and field skid test equipment (6).

Research was undertaken to evaluate the problem of wear and polish of highway pavements and their effect on the pavements' skid-resistance characteristics. A wear machine was developed which applied a controlled load against a pavement sample through a reciprocating rubber pad. The pavement samples consisted of aggregates glued to a metal holder by an epoxy cement. A mixture of water and abrasive was injected periodically between the surfaces in contact. Wear as such was not measured, but the polish produced by the wear was determined with the British pendulum tester. The general conclusion derived from the study was that the basic wear mechanism must be better understood and wear produced under conditions more closely resembling those on the highway must be studied before reliable results can be expected from the greatly accelerated wear which the reciprocating wear machine produced (7).

Other basic skid research conducted under this program which may be of interest, although not directly applicable to the scope of this paper, includes studies on the concept and use of the British portable tester (8), pavement wetting and skid resistance (9), pavement surface characteristics (10), and mineral wear in relation to pavement slipperiness (11).

THE SKID SURVEYS

Through an early phase of the Joint Road Friction Program, the Department was able to obtain its own single-wheel skid trailer in time to conduct a statewide annual skid survey in the fall of 1963. This, and all subsequent annual surveys through 1967 were conducted solely by the Department.

Responsibility for the selection of test sites is given to each of the 11 engineering districts in Pennsylvania. In the spring of each year the Department solicits from each district headquarters a list of 15 to 20 bituminous surface pavements where verification of slippery conditions is required. Most of the pavements thus listed are well-worn but many have only been in service a few years and some are new, so that the data are nearly representative of all type and ages of surfaces. It should be emphasized, however, that these pavements were listed because of a suspected slippery condition, so that, overall, the results may not truly reflect a cross-section of Pennsylvania's highways with respect to skid resistance.

The skid test crew begins testing as soon as the lists of pavements are received from the districts, which is usually in June or July. This is a departure from the original plan to confine testing to the fall season. Originally it was recommended by Penn State that the late fall was the best time to determine accurately the skid resistance of pavement, the reason being that this was the driest season and skid readings would be expected to be at their lowest. It was not expedient, however, for the Department to restrict thousands of miles of testing to a short, one- or two-month period in most areas of Pennsylvania. It was decided, therefore, to conduct year-round testing as weather permitted. The reasons were conscientiously considered: (a) other skid research programs were being given least priority and relegated to the "worst" testing season; (b) realistic skid readings for the high-volume summer traffic season could be recorded and compared from district to district as more years of data were gathered; and (c) seasonal effects on a pavement's skid resistance could be studied.

Survey data are returned to the individual district offices as each district's testing is completed, with recommendations for remedial action in specific cases. Some months later, a complete statewide survey report is prepared for the central office engineering staff. This report contains a compilation of data by district and an analysis and cumulative totals of all data to date. These annual reports have not been released, but some of the results are presented in this paper.

In completing the five annual surveys, the test crew logged nearly 25,000 miles, exclusive of other research, and averaged about 150 test projects per year. To accomplish this the test wheel has been dragged over the surface in a locked position for an estimated 500 miles. Considering all expenditures such as salaries, subsistence, and vehicle costs (including depreciation), it has been calculated that the average project costs $30.00 to test (based on 15 test cycles per project).

EQUIPMENT AND TEST PROCEDURE

The PDH Road Friction Trailer (Fig. 1) is an improved model of the original Penn State Road Friction Tester and was constructed under contract by the Mechanical Engineering Department of the University (5). The major change from the prototype, and now incorporated in it also, is the wheel force conversion system from a totally electrical system to a nearly totally hydraulic system whose response time is adequate (12).

During testing, the truck is operated at 40 mph for a distance adequate to appraise the whole pavement. The tester is designed to sample automatically, and at regular intervals, about 10 cycles per mile at 40 mph. The randomness of the locked-wheel intervals provides fair and adequate representation over the project. The skid number (defined as the friction coefficient of a tire skidding on a wet pavement times 100), as obtained from the left wheel path of the traffic lane, is recorded on a strip chart in the truck cab (Fig. 2).

A secondary field tool, known as the Penn State "drag tester" (13) (Fig. 3), accompanies the skid truck and is used in areas where the truck cannot operate safely. The operator merely samples the wetted pavement by pushing the tester at a normal walking speed of about two to three mph. The good to excellent correlation with a locked-wheel tester indicates the ability of the drag tester to serve as a substitute for a skid trailer whenever the latter cannot be used. The excellent correlation between the drag tester and the British pendulum tester indicates that the two devices are equivalent as far as the quality of test data is concerned (14).

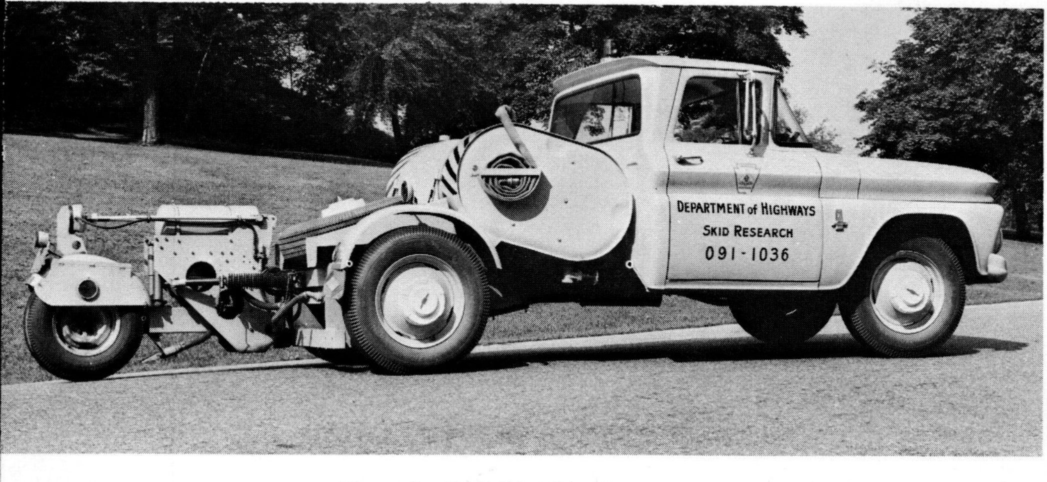

Figure 1. PDH road friction tester.

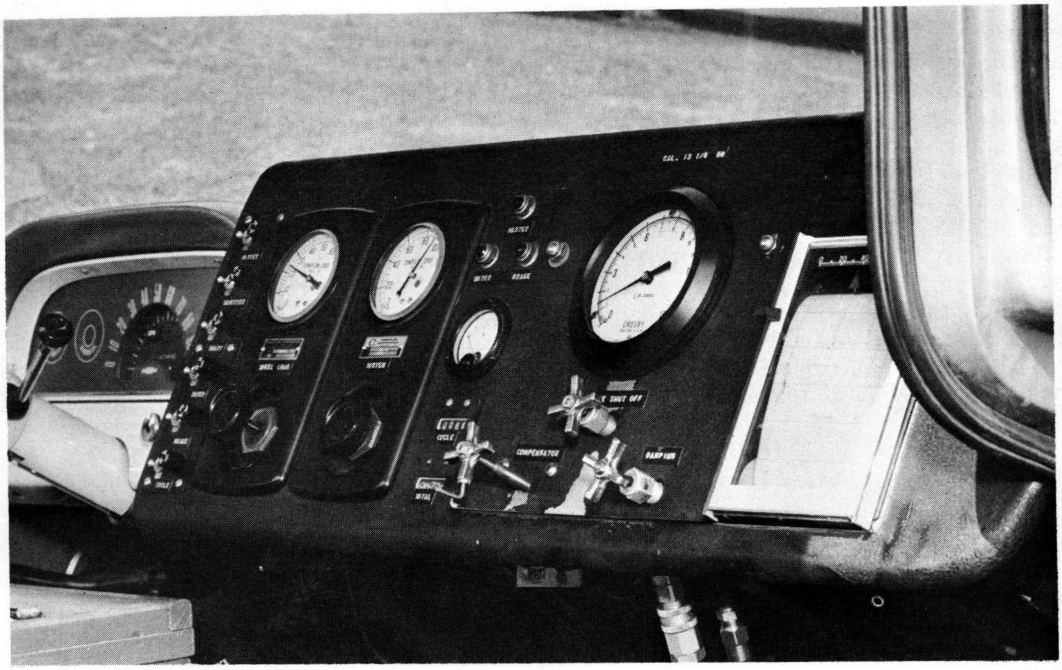

Figure 2. Instrumentation.

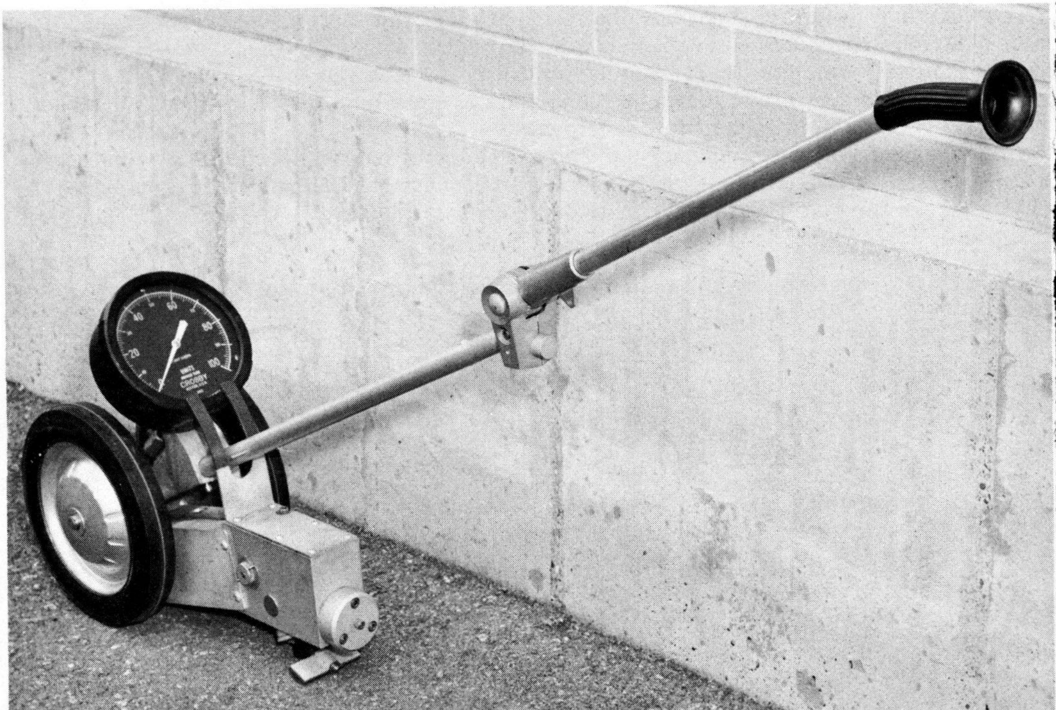

Figure 3. Drag tester.

SKID RESISTANCE SURVEY
FIELD RESEARCH DATA

L. ROUTE ______________ COUNTY

T. ROUTE ______________ DISTRICT ______________

STATIONS		TYPE SURFACE	TYPE AGGREGATE	AGGREGATE PRODUCER	EST. ADT	YEAR PLACED	NO. PASSES (x 1000)	SKID READING (HI/LO AVG)	NO. CYCLES	DATE	AIR TEMP	REMARKS
START	FINISH											

(FRONT SIDE)

DATE	QUALITATIVE ANALYSIS														
	FI	MD	CO	OP	CL	SM	RO	PO	GR	FA	BL	PA	RI		

LEGEND:
- FI FINE-TEXTURED SURFACE
- MD MEDIUM-TEXTURED SURFACE
- CO COARSE-TEXTURED SURFACE
- OP OPEN-TEXTURED SURFACE
- CL CLOSED-TEXTURED SURFACE
- SM SMOOTH SURFACE
- RO ROUGH SURFACE
- PO POLISHED SURFACE
- GR GRITTY SURFACE
- FA FATTY SURFACE
- BL BLEEDING SURFACE
- PA PATCHED SURFACE
- RI RIPPLED SURFACE (WASHBOARD)
- SY SLIGHTLY
- PY PARTLY
- VY VERY

(REVERSE SIDE)

Figure 4. Field data sheet.

Pertinent field data are recorded on the skid survey form (Fig. 4). Surface temperature of the pavement, though not listed, is measured and recorded under "Remarks." Qualitative measurements are recorded on the reverse side of the sheet and include such parameters as roughness, polishing, openness, grittiness, harshness, fattiness, extent of bleeding, degree of patching, and rippling features. A one-through-three designation denotes progressively more of each parameter's presence. No comparative analyses of these parameters with the skid number have been made.

SURVEY RESULTS

An analysis, however, was made of the more obvious parameters such as aggregate type and surface type. The data indicate that aggregate type bears much influence in determining the skid resistance, whereas surface type has very little influence. These data are presented in the form of bar graphs which compare the aggregate type versus skid number (Fig. 5) and the surface type versus skid number (Fig. 6). The data are based on information gained from the skid surveys during the period from 1962 through 1967. The mix designations, characteristic to Pennsylvania, are identified as follows: ID-2 is a two-aggregate composition wherein the coarse aggregate passes the ½-in. sieve and the fine aggregate is a typical bituminous concrete sand passing the 3/8-in. sieve; FJ-1 is a sand-asphalt mix using the same bituminous sand gradation; FJ-3 is a sand-asphalt mix with synthetic rubber additive; FJ-4 is the same mix as the FJ-1 with an asbestos additive; S. T. designates a surface treatment, sometimes referred to as a chip seal.

Figures 5 and 6 were compiled from data collected on 773 separate roadway projects. The numbers in parentheses reflect the total number of projects tested in each category. In Figure 5, despite the variation in the average skid number between aggregate groups, the surface types within each group fluctuate but little. There is a pronounced degradation in skid resistance in descending order from gravel and sand down to limestone.

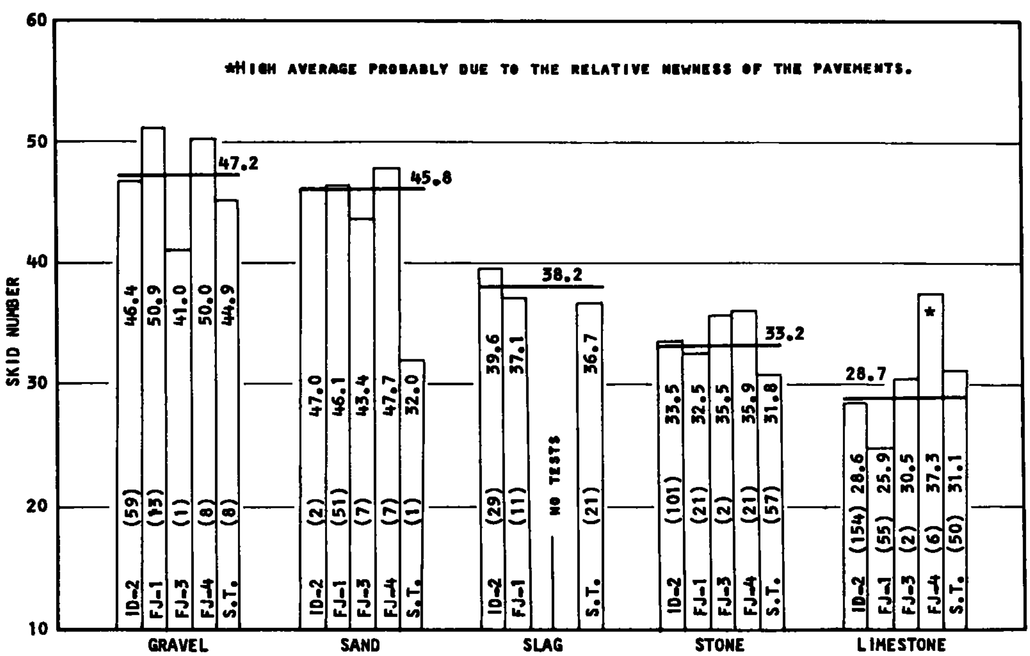

Figure 5. Effect of aggregate type on the skid number, values shown for various surface types are weighted averages for all tests recorded 1962-1967, numbers in parentheses represent the number of tests.

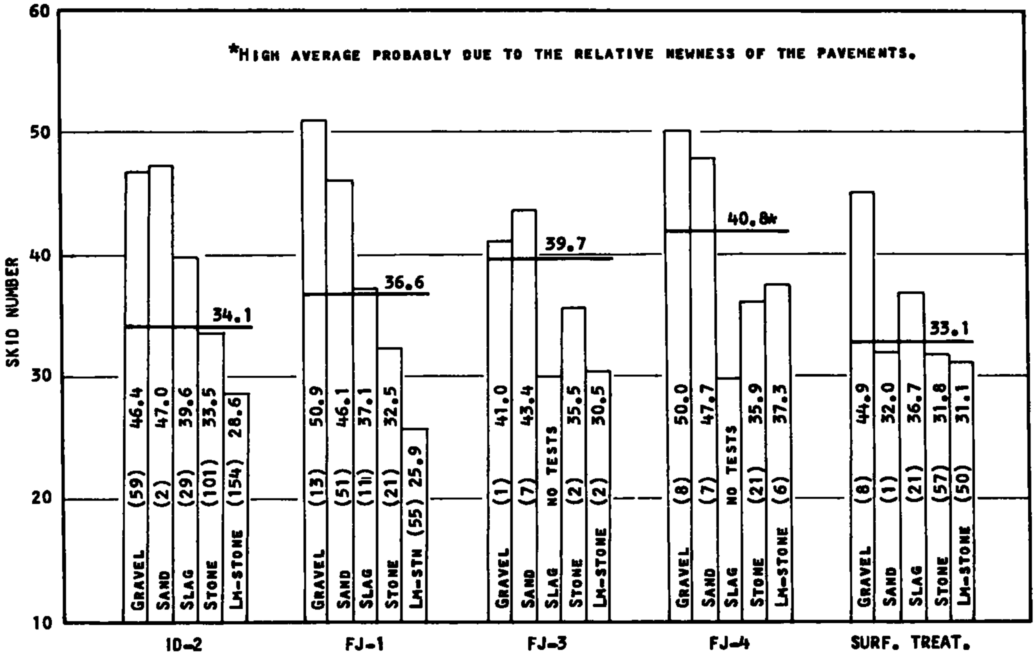

Figure 6. Effect of surface type on the skid number; values shown for various aggregate types are weighted averages for all tests recorded 1962-1967; numbers in parentheses represent the number of tests.

Conversely, in Figure 6 the average skid number of each surface type group varies little from one to another. However, within each group the variation is extreme, due to aggregate influence. The pattern follows the same descending order depicted by Figure 5. This information is not startling in terms of present knowledge but it confirms on a more substantial basis what has been suspected from previous but limited research. Although the data have been obtained from roadway surfaces suspected of slipperiness, and the results verify this, it is sincerely felt that these data could be projected validly to reflect a similar pattern if a random selection of pavements throughout the state were tested.

CURRENT PROGRAM

In-House Research—The Department is satisfied with the accomplishments of the surveys in that they have provided valid information regarding the skid resistance of various types of aggregates and surfaces and have served to establish and improve testing methods and techniques for research purposes. They have also made the districts aware of the importance and necessity of locating slippery pavements and taking the necessary corrective measures. These decisions have been based generally upon recommendations from the Bureau of Materials, Testing and Research and the acceptance of a minimum Skid Number of 40.

The next phase, pending delivery of another skid trailer, is to expand the skid survey program and inaugurate a full-scale, statewide roadway inventory. To expedite this, a third skid trailer will be acquired during fiscal 1968-69. The task will be tremendous, even if sampling techniques are employed, considering the more than 43,500 miles of highways on the state system. With three separate crews functioning, one truck will be assigned to purely research work while the others will perform service testing and road inventories.

The capability of the present road trailer will be increased considerably after installation of paper punch tape for data acquisition. A magnetic instrument tape recorder was not suitable for this vehicle, due to space and shock mounting requirements. Future vehicles will be equipped for total automatic data acquisition, however, including 7-channel magnetic instrument tape recorders. Automatic data processing equipment will be utilized to expedite data analysis. Provisions for data storage and recall have been initiated and should be functional in the near future. The use of portable skid measuring equipment in each district will be considered and the skid trailers may be assigned on a regional basis. The surveys will continue to provide the service function to the engineering districts as in the past; but because of more local assignment, the crews will be able to provide closer control of the data acquisition. At the same time, recording of all data on magnetic tape will permit the central research unit to process the input easily and promptly.

Experience gained while conducting the annual surveys has enabled the Department to undertake better-controlled research projects. An example of this is an aggregate blend study designed to show the influence of various amounts of skid-resistant aggregate in limestone mixes and evaluate the durability of bituminous mixes having various proportions of gravel. Recognizing that the supply of skid-resistant aggregates is critical in many areas of Pennsylvania, the Department initiated a field research project in an attempt to upgrade the skid resistance of local, polish-susceptible aggregates (15). The project was a surface-treatment-type application which included limestone, slag, and crushed and uncrushed gravel of several gradations. The slag and gravel sections produced skid numbers almost twice as large as the crushed limestone sections. An admixture of 25 percent by volume of slag and gravel to limestone surface increased the skid resistance slightly, but not enough to offset the inherent slipperiness of the principal aggregate component. Additional studies are being planned to evaluate further the effectiveness of blended aggregates.

Another research project is being conducted by the Department in one of the local engineering districts. The skid resistance histories of 45 new pavements are being kept to note their rate and pattern of deterioration. The program is designed to relate the effects of external variables such as temperature, season, and traffic to the numerous parameters of the road surface such as texture, aggregate and surface type, and asphalt content. Data are being collected on approximately 57 pavement sections. Of this total, 45 are tested on a monthly schedule while the remainder are tested annually or semiannually. Several of the special study groups such as the fine aggregate blend study are tested on a monthly schedule. The observations to date parallel the conclusions formulated previously from the skid surveys. Of particular interest are the specific decline curves for the particular aggregate combinations showing skid resistance degeneration over a period of time with traffic.

The need to record parameters other than those shown on the field survey data sheet has resulted from the annual survey. Accurately determined pavement characteristics (aggregate size distribution, aggregate mineralogy, cement concrete petrography, asphalt content, and others) are requisite to skid research, and a sampling program in conjunction with the testing has been initiated by the Department. The purpose of the program is twofold: first, to provide precise parameter determinations which relate to skid numbers; and second, to provide a means of perfecting a laboratory method for determining accurate skid-resistance levels of actual pavement surfaces. To accomplish this, the Department has assigned a core drill to the skid research unit and when expedient, all tested surfaces are cored, oriented, and carefully returned to the laboratory for skid testing with the British portable tester. A core holder (Fig. 7) has been devised to hold 4- and 6-in. cores in a level and properly oriented position and to provide an elevated stand for the British portable tester. A useful correlation between the British portable tester and the skid truck data, based on thousands of samples, should determine the feasibility of developing a British portable skid-resistance scale as a means of determining acceptability or rejection of a pavement.

Certainly, additional studies are urgently needed to determine the influence on skid resistance of such things as the chemicals placed onto a surface in the winter season, the effect of season and surface temperature, and the mineralogy and petrology of the aggregates themselves. The Department's petrographic laboratory is conducting a program to determine the relationship of skid resistance to the petrography of the

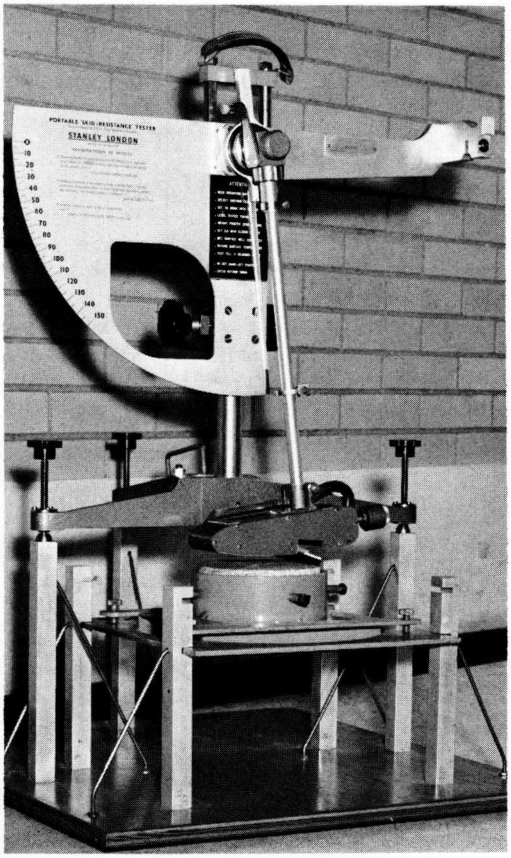

Figure 7. Core holder for RPT.

aggregates. For each general type of aggregate there is a wide range of skid resistance and the controlling aggregate parameters have not been clearly identified except in certain specific cases. At the present time there is no satisfactory construction or material specification to insure that a pavement surface will have adequate skid resistance after exposure to a considerable volume of traffic. The objective of this research program is to correlate the petrographic properties of aggregates used in roadway surfaces with the road friction tester skid measurements for those same sections of roadway. Further, it is hoped that a relationship can be established between the skid-resistant parameters of the aggregates and the identifiable stratigraphic units of both present and potential aggregate sources.

Continuing Sponsored Research—The Department will continue its sponsorship of the Joint Road Friction Program at Pennsylvania State University. The research is being directed under two phases: pavement slipperiness and pavement polishing. The objective of the former is to deepen the knowledge of the mechanism of friction between tire and pavement, and to employ this knowledge to develop guides for the construction and maintenance of pavement surfaces with improved frictional characteristics. Continued tests with the road friction tester will be concerned basically with a comparison of the ASTM tire with representative commercial tires. The research on the relation between pavement characteristics and tire-pavement friction will be continued, the most important phase of this effort being the investigation of aggregate shape effects. The measurement of pavement texture from a moving vehicle will be pursued further as will modifications of the road friction tester permitting it to operate in the slip mode.

The research on pavement polishing will investigate the polishing characteristics of selected aggregates and the effectiveness of roughening techniques. An attempt will be made to devise better methods of relating laboratory and field polishing data. A rotary wear machine has been constructed and work will continue on the study of aggregate polish with the use of roughening agents. A study will be undertaken to determine the best design for a polishing apparatus equally suitable for use in the laboratory and on the highway. The nature of the debris on the highway as it relates to polishing will receive further attention.

Implementation of Research—The research brought about by this program, together with the implications of the Federal Highway Safety Act, have created an immediate sense of awareness of the skid problem by the Department's top-level engineering staff. A directive from the Secretary of Highways to all engineering districts has advised them of the Department's intent to issue supplements to the Specifications providing for highly skid-resistant surfaces for both concrete and bituminous pavements. The program will be implemented in 1968 for projects under contract and will be extended in 1969 to cover surface treatments and wearing surfaces placed by the Department's maintenance forces.

The Bureau feels that the selection of an initial skid number and the enforcement of such specifications would be difficult. Current data on skid resistance are largely related to pavements which have been in service for some time. The histories compiled on road surfaces prior to traffic exposure are scarce. Limited data indicate that initial skid values are often high. The skid number can decrease by 40 percent in a few months when the aggregate is a polish-susceptible type.

Records have shown a wearing surface to read as high as 60 initially and later drop below the 40 Skid Number level. If a Skid Number specification were in effect, the Department could unintentionally accept a pavement that might later prove hazardous. If a high initial Skid Number is arbitrarily selected, certain wearing surfaces which may maintain a value of 40 or more for their service life, might not be able to qualify initially. Data indicate that these possibilities exist. Sufficient information to establish an initial Skid Number specification in Pennsylvania is not available at this time.

The Department is considering adoption of a supplement to the specification that would provide aggregate qualifications and the texturing of concrete rather than a Skid Number. The following recommendations are being considered:

1. Bituminous concrete:
   - Fine Aggregate: Fine aggregate produced by the crushing of carbonate rocks shall not be used in bituminous wearing surfaces, but may be used in the base and binder courses.
   - Coarse Aggregate: Aggregate prepared from carbonate rocks containing less than 25 percent siliceous particles shall not be used in bituminous pavement wearing surfaces but may be used in the base or binder courses and in the shoulder wearing surface. The siliceous particle content shall be determined by the Pennsylvania Department of Highways Test Procedure No. 203-68, Solubility Test for Aggregates.
2. Portland cement concrete:
   - Fine Aggregate: Sand produced by crushing carbonate rocks shall not be used in concrete wearing surfaces.
   - Final Finish Requirements: After the straightedge testing and surface corrections have been completed and just before the concrete becomes nonplastic, the surface shall be given a drag finish by dragging a seamless strip of damp burlap, cotton fabric or a wire brush longitudinally along the full width of pavement which shall produce a uniform surface of gritty texture. The texture produced shall have a minimum surface relief of 0.025 in. The texture of the surface shall be of uniform appearance and reasonably free from grooves over $^1/_{16}$-in. depth.

SUMMARY

The statewide annual survey was a wise choice in initiating a skid resistance research program in Pennsylvania. The Department has gained invaluable experience in technological know-how, as well as much knowledge with respect to the factors influencing skid resistance and pavement performance.

Aggregate type appears to be quite influential in contributing to skid resistance but the surface type into which the aggregate is mixed has little or no significance. Gravel and natural sand (quartziferous) aggregates produce pavements with more skid resistance than do limestone (calcareous) aggregates. Bituminous mixes made with some of the quartziferous aggregates, especially some of the gravels in Pennsylvania, create problems in other respects, however. Many of the flexible, gravel-aggregate pavements tend to ravel or lose aggregate.

The initial skid readings may be misleading if taken too soon. Asphalt coating of the aggregate definitely can influence its performance under the test wheel. Consequently, the Department is pursuing means to age samples of new pavement surfaces artificially in order to predict, by testing methods, the approximate level of skid resistance after a given number of passes, not yet determined.

The Department's pursuits in skid resistance research include many areas, but all contribute to the primary purpose, which is to provide a safer roadway on which to travel.

ACKNOWLEDGMENT

The authors wish to express appreciation to Professor W. E. Meyer and staff of The Pennsylvania State University for their contribution of background information to this report.

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Discussion

J. PAUL MARTIN and L. JOHN MINNICK, Technical Committee of the Pennsylvania Stone Producers Association—This discussion is presented because we believe, as members of a very important industry, that our thoughts and opinions should also be heard in this very vital matter. First of all, stone producers want to acknowledge that a problem does exist in many areas and we realize that aggregates are a part of this problem. As producers of aggregate we are also businessmen who wish to protect our industry from invalid or improper restrictions in use, especially since this industry is a vital segment of the economy of the country. We are interested in the development of realistic and meaningful aggregate specifications, but it is contended that the data presented by the authors do not warrant the extremely restrictive specification which they have suggested.

Pennsylvania is the leading producer of stone in the nation and has at the present time approximately 223 operating stone quarries and 115 sand and gravel pits. Most of this stone is supplied to the construction industry. Of all of the aggregate used in construction, approximately 75 percent represents stone. The type of stone varies from carbonate rocks, including calcitic limestone, magnesian limestone, and dolomite to siliceous materials such as diabase, sandstones, and gneiss. These aggregates have been used for many years in the construction of wearing courses on roads and considerable experience is available as to their performance in a variety of asphaltic and portland cement concrete mixtures.

The paper discusses the performance of some of these wearing courses in terms of the effect of both aggregate type and mix composition on the skid resistance properties of the pavement.

The data are based mainly on a series of statewide surveys which has been made on pavements that were "selected because of a suspected slippery condition." Each of these surveys has noted that this method of selection will give unrealistic or biased answers. Quoting from the 1965 Survey, "In most cases, only potentially slippery pavements were tested; therefore the average does not necessarily reflect a cross-section of the Commonwealth's highways" (16). From the 1966 Survey, "Since the Bureau specifically requested that the Districts submit a list of potentially slippery pavements..., the low, statewide skid average of 37 does not reflect an accurate cross-section of the Commonwealth's highways" (17). The 1967 Survey recognizes this problem and states further: "District and statewide averages would be unfair and in some cases meaningless, since the test projects submitted were, for the most part, subjectively selected" (18).

In spite of these observations the authors have, based on the self-same data, made numerous general conclusions pertaining to the effect of aggregate type and mix design on the skid resistance of bituminous pavements. They have stated: "Although the data have been obtained from roadway surfaces suspected of slipperiness, and the results verify this, it is sincerely felt that these data could be projected validly to reflect a similar pattern if a random selection of pavements throughout the state were tested" (19).

Thus, in spite of the fact that the individual surveys have each indicated that statewide averages are biased and in some cases meaningless, the authors have persisted in projecting general conclusions based on their feelings about these averages. The conclusions have been extended into a projection of overall performance of pavements that have not even been included in the survey at all. It is obvious that this type of extrapolation is totally unscientific and any accuracy of the conclusions obtained would be merely fortuitous, regardless of the number of tests involved in the study.

To illustrate the uncertainty of such an approach as utilized by the authors, the data available in the surveys have been statistically evaluated by computer and some of the results of this evaluation are presented.

Frequency of Testing

Based on the sampling plan which was used in the Pennsylvania Survey, the frequency of sampling with respect to mix or aggregate type serves as an indicator of the

TABLE 1

RATE OF TESTING

Mixture	ID-2	JA-1	FJ-1	FJ-2, 3, 4	FB-1, 2, 3
(a) Mixtures Placed					
Mixtures used in 1957-1966 (as percent of total bituminous concrete tonnage)	84.4	2.9	8.2	0.3	3.1
(b) Mixtures Tested					
Mixtures tested in 1962-1967 surveys (as percent of tests in Figure 6)	50.2	–	22.0	7.9	–
Mixtures tested in 1966 (as percent of tests in 1966 survey)	42.4	4.2	27.3	10.9	4.8
Mixtures tested in 1967 (as percent of tests in 1967 survey)	45.8	0.0	31.9	4.9	0.7

performance of a particular mix or aggregate. Any mix that performs well will be sampled infrequently, and conversely, any mix that does not perform well will be sampled more frequently than would be indicated by the amount of this material that was placed in service.

A review of the paper as well as the 1966 and 1967 Surveys shows that certain mixes are tested at a rate which is less than proportional to the amount of these mixes in service, indicating that these mixes are producing a disproportionately low number of slippery pavements (Table 1). ID-2 mixes are tested at a rate equal to about one-half of the rate at which they are placed, whereas FJ-2, 3, and 4 mixes are tested at 26 times the rate at which they are placed. This clearly indicates that, contrary to the authors' conclusions, mix design is extremely important as a factor in controlling skid res stance in Pennsylvania.

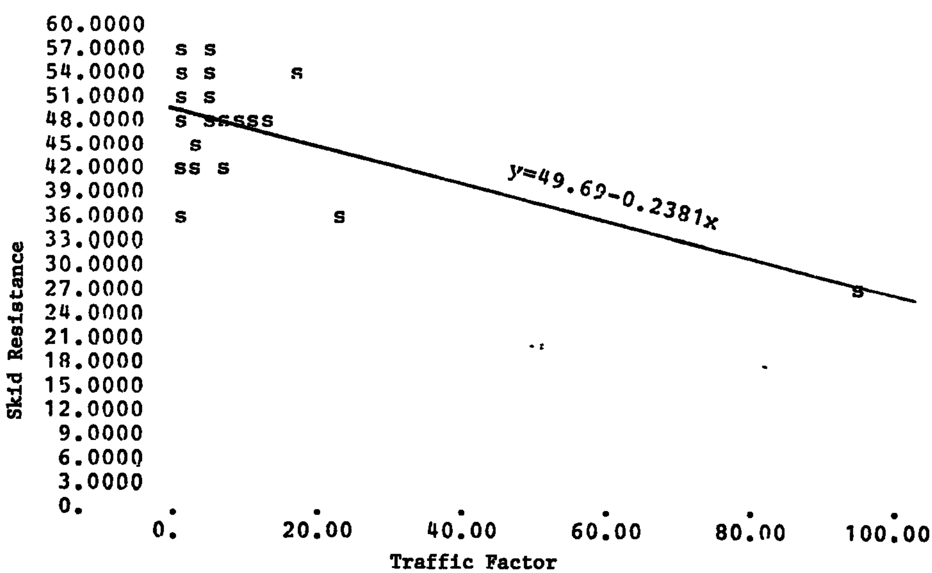

Figure 8. Pavements containing sand.

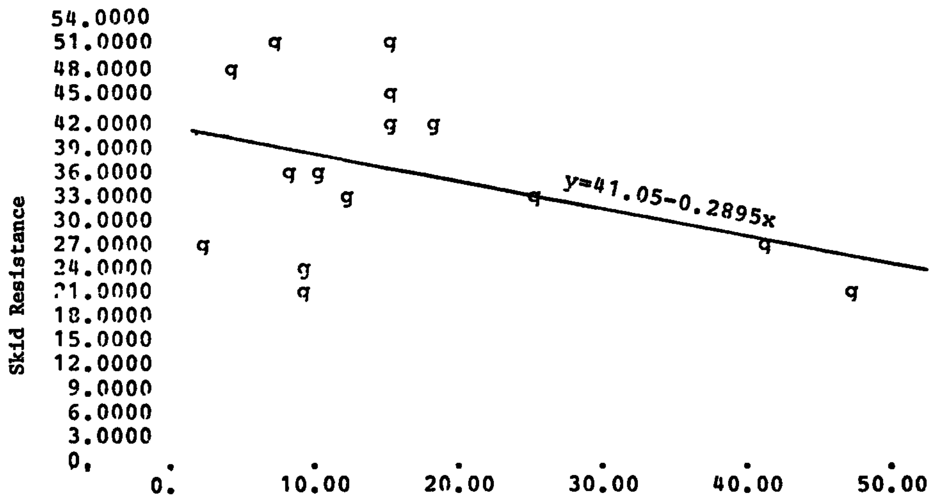

Figure 9. Pavements containing gravel.

Similar reasoning based on the amount of stone and limestone used and tested in Pennsylvania indicates that limestone may actually be performing better than sand and gravel. For example, in particular districts in the state where the majority of the pavements are made with dolomitic limestone, very little testing has been required; this indicates that these pavements are performing quite well. These results compare well with the report by Shupe and Lounsbury, which showed that dolomites exhibited up to 75 percent greater skid resistance than high calcium limestone (20). No attempt was made in the Pennsylvania surveys to differentiate between these two types of stone.

Rate of Polishing

Figures 8 through 11 present traffic profiles for the four types of aggregate referred to in the Pennsylvania Survey. The plots have been made by computer and regression lines (also calculated by computer) are added to the plots.

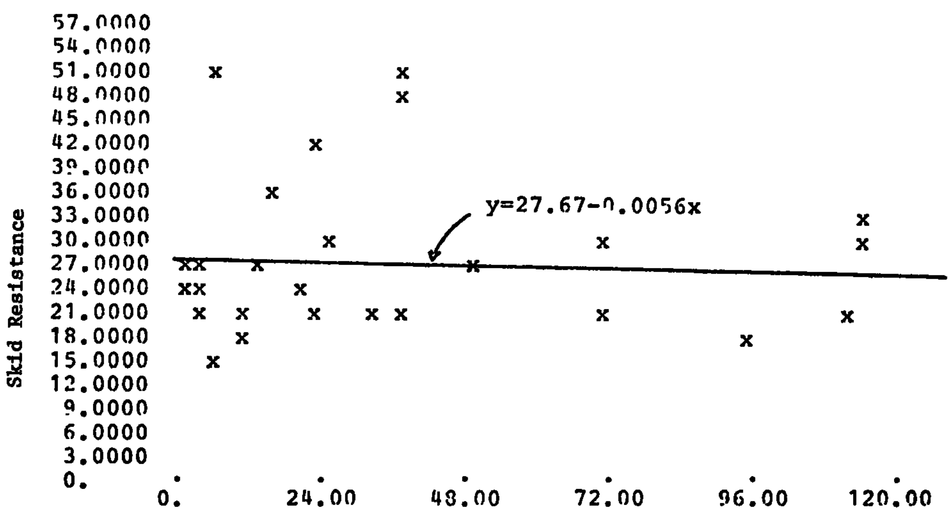

Figure 10. Pavements containing stone.

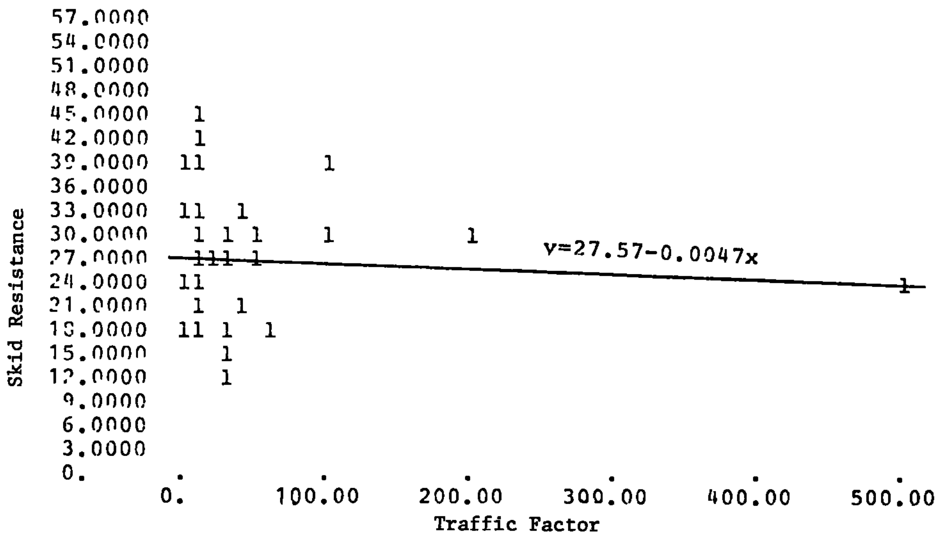

Figure 11. Pavements containing limestone.

These graphs indicate that stone and limestone are polishing only to a negligible degree, and that these materials will produce in service regardless of initial skid values pavements which are superior to those obtained with sand and gravel. Work in Michigan also observed that degradation in skid resistance occurred at a faster rate with gravel than it did with limestone (21).

Traffic Conditions

It should be noted that the stone and limestone were tested under traffic conditions which were three to four times as severe as those of sand and gravel in the 1967 Survey (Table 2). Also, in the 1966 Survey, similar differences occurred. The oldest gravel pavement tested was placed in 1954; the oldest limestone pavement was placed in 1932, and 6 limestone pavements were placed in the 1940's. It is significant that the limestone surfaces have remained in service for this length of time, whereas the gravel surfaces have presumably required resurfacing.

When the skid values which were obtained for each aggregate in the 1967 Survey are adjusted to the average traffic factor of the limestone pavements (Table 3), or to a traffic factor of 100, the skid values which are established for each aggretate show that limestone will perform as well as or better than sand and gravel in mature pavements.

These statistical evaluations of the Pennsylvania Surveys obviously present conclusions which are different than those submitted by the authors and thus clearly indicate the lack of validity of the conclusions.

As a further point of concern to the Pennsylvania stone producers, it is noted in the authors' presentation that the selection of test sites was developed each year in each of the engineering districts of Pennsylvania and that a list of 15 to 20 pavements were involved in each district. It is to be noted that there were exceptions to this plan and a number of the districts were not properly represented because there were not sufficient examples of slippery pavements to warrant an investigation. For example, the 1967 Survey includes only 4 sites in District 6.

The Pennsylvania stone producers have conducted tests using the Penn

TABLE 2

TRAFFIC CONDITIONS

Aggregate Type	Traffic Factor Min.	Max.	Avg.
Sand	0 03	94 91	11 30
Gravel	1.16	45.57	14 75
Stone	0.68	113.40	33 01
Limestone	1 43	495.00	46 71

TABLE 3

ADJUSTED SKID VALUES

Aggregate Type	Skid Resistance of Pavements Tested in 1967 Survey	
	At Traffic Factor = 46.7	At Traffic Factor = 100
Sand	38.6	25.9
Gravel	27.5	12.1
Stone	27.4	27.1
Limestone	27.4	27.1

State drag tester in this same District on over 70 pavements; these are recognized as being pavements that are performing normally. They range from 1 to 10 years in age. The findings show many examples of pavements that are performing unusually well under high traffic conditions that utilize types of aggregate that are given poor ratings in the paper. For example, the skid values for ID-2 mixtures containing dolomite as the aggregate were among the highest for pavements that have been in service for more than 5 years.

These findings serve to emphasize again the invalidity that results from projecting the data of the Pennsylvania State Surveys into a prediction of the anti-skid performance of bituminous wearing surfaces in general. In view of this, it is unfortunate that the authors attempt to develop a blanket specification at this time, particularly since the Pennsylvania Highway Department has established a good research and field program that should give more realistic information in the near future. As an example, in the Department's current investigation, several interesting posibilities are being explored in connection with the use of aggregate blends and other changes in mix design which have indicated improved skid resistance characteristics.

Again, we wish to emphasize that we as a very vital industry do not want to find an unjustified all exclusive specification eliminating many quarries which are producing completely satisfactory pavements. It is our belief that blending operations are possible and have been used satisfactorily in other states. A joint task force of industry and Department of Highways' representatives is evaluating this and other avenues of approach at the present time.

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Recommendations for an International Minimum Skid-Resistance Standard for Pavements

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The need for minimum international skid-resistance requirements on roads and highways is apparent from accident statictics in several countries. The interpretation and implementation of any such standards, however, are made difficult by the many variables present with existing test procedures. These include test tire parameters, environmental factors, and particularly the mode of testing (locked-wheel, stopping-distance, deceleration-sensing, brake and side-slip).

It is recommended that a meaningful international minimum standard should specify minimum coefficients of friction irrespective of test method and at two distinct speeds, perhaps 50 and 90 km/hr. Previous work has suggested that the coefficient be specified at two distinct speeds, since pavements which perform similarly at one test speed may have totally divergent coefficients at other speeds. Specifying that these minima are irrespective of test method is an essential criterion for eliminating variations, and is of course a more practical approach since a skid may occur under any condition of braking or cornering. It is believed that this objective can be met with the use of correlation plots between different test modes, such as those already established in Sweden and the Netherlands. Future test programs can serve to establish a comprehensive listing of correlation factors for the successful implementation of any international standard.

•AN inverse relationship between skid resistance on wet roads and accident rates from British statistics (1) illustrates the importance and need for the maintenance of high skid-resistance values. Dutch figures (2) suggest that about 1 percent of all road accidents involve death and 14 percent serious injury, while again British results (3) indicate that the number of accidents involving skidding on wet roads is about 30 percent of all accidents occurring in wet weather. There is therefore both a need to improve the friction potential of highways in general and to establish minimum acceptable skid-resistance values.

The mechanics of the onset of skidding on wet surfaces is now broadly understood (4, 5, 6) and it is possible to select the features of surface texture so that skid hazards are minimized or even eliminated on certain pavement sections (7). The measurement of effective traction, however, presents many problems which have been hitherto unresolved. These problems arise from differences in the mode of testing (whether locked-wheel, stopping distance method, brake or side-slip), the test tire used (size, type, pattern), the design of tester selected, test speed and other environmental factors. Indeed, while there may be a consensus that a certain pavement section is excessively slippy, the exact value of its "slipperiness" as suggested by its measured coefficient of friction is unknown. The high accident rates on wet road surfaces recorded in the above statistical data point to the need for establishing universal minimum standards of acceptable slipperiness, but the anticipation of what these standards might mean and their subsequent interpretation presents difficulties because of the variability in measurement techniques. It has already been established (7) that either two sufficiently different test speeds must be selected for road surfaces, or alternatively one test together with a measurement of skid-resistance/speed gradient.

EXISTING INTERNATIONAL MINIMUM STANDARDS

Figure 1 shows existing international minimum coefficients of friction plotted against test speeds (8, 9, 10). The locked-wheel, stopping-distance and brake slip modes of testing are included for patterned test tires and the side-slip mode for smooth tires. The most significant standards are those suggested by the Permanent International Association of Road Congresses, who recommended a coefficient of friction of 0.45 at 45 km/hr (later 0.40 at 50 km/hr) measured using any test mode.

For historical and other reasons, different countries have been prone to select one basic method of measurement in preference to others. Thus, the British (8) insist on measuring sideforce using a smooth tire with slip angle, and the Swedes (8) measure almost exclusively with a braked wheel and 15 to 17 percent brake slip using a patterned tire. Perhaps the method of locked-wheel braking is the most popular overall method, but in the United States (8) stopping-distance and deceleration-sensing techniques are widely used although almost unknown in Europe. Despite these differences, the minimum coefficients as plotted versus test speed indicate broadly what is and what is not acceptable.

In attempting to reduce the obvious discrepancies which must arise from using different methods of measurement, the Swedes give the following relationships between locked-wheel and brake slip coefficients (8):

$$\text{Concrete Roads:} \quad f_{LW} = 0.7\, f_{BS,\,17\%} - 0.048 \tag{1}$$

$$\text{Asphalt Roads:} \quad f_{LW} = 0.686\, f_{BS,\,17\%} + 0.016 \tag{2}$$

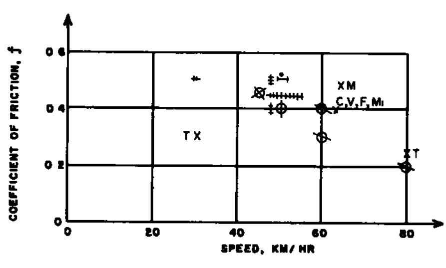
while the Netherlands State Road Laboratory (8) reports that using patterned tires:

$$f_{BS} = 1.35\, f_{LW} \quad \text{at 20 km/hr} \tag{3}$$

and

$$f_{BS} = 1.15\, f_{LW} \quad \text{at 50 km/hr} \tag{4}$$

These attempted correlations are most useful in establishing minimum international skid-resistance standards.



SYMBOL	TEST MODE	ORIGIN OF STANDARD	GENERAL NOTATION
⊠	ANY	9TH INTERNATIONAL ROAD CONGRESS	C = CALIFORNIA, (P)LW V = VIRGINIA, (P)SD
⊕	ANY	10TH INTERNATIONAL ROAD CONGRESS	Mi = MISSISSIPPI, (P)SD F = FLORIDA, (P)SD
•	(P) BS	DUTCH STATE ROAD LABORATORY	M = MICHIGAN, (P)LW T = TEXAS, (P)LW
⊘	(P)LW	GERMAN	
⊘	(P)LW	PROPOSED GERMAN	(P)LW = PATTERNED TIRES LOCKED WHEELS
++	ANY	FRENCH	
⊢⊣	(P)LW	ITALIAN	
+++++	(S)s	BELGIAN	(P)SD = PATTERNED TIRES STOPPING DISTANCE
∓	(S)s	BRITISH MINIMUM (STRAIGHT ROADS)	
‡	(S)s	BRITISH MINIMUM (GENERAL)	(P)BS = PATTERNED TIRES BRAKE SLIP
X	(P)LW or (P)SD	AMERICAN	(S)s = SMOOTH TIRES SIDE-SLIP

Figure 1. Existing international minimum coefficients of friction.

RECOMMENDATIONS FOR INTERNATIONAL MINIMUM SKID-RESISTANCE STANDARD

It is a well-known fact that the coefficient of sliding friction on a wet road surface decreases with increase of speed in a manner which depends almost exclusively on the size of surface texture. Thus, two pavements (one fine and one coarse) may have the same coefficient of sliding friction at 50 km/hr under wet conditions—but the values at 80 km/hr will be entirely different. The measurement of friction at one speed value is therefore inadequate to characterize pavement performance. It is therefore recommended that a meaningful international minimum standard should specify the minimum coefficient irrespective of test method at two distinct speeds (perhaps 50 and 90 km/hr).

Specifying that these minima are irrespective of the method of measurement is an essential condition for eliminating variations, and of course is a more practical approach since a skid may occur under any condition of braking or cornering. Before this can be accomplished, it will be necessary to specify equations of the type of Eqs. 1 through 4 showing relationships between frictional coefficients obtained for a variety of speeds, road surfaces, and including all modes of testing. Of course, much work remains to be done to propose a universal set of correlation plots, but once completed the establishment of an international standard is a relatively easy matter.

Given an absolute minimum coefficient as criterion at a particular speed, information obtained from these correlation plots will tell which method of testing should yield the minimum coefficient for any type of surface at a particular speed. Assuming that another mode of measurement is used for routine testing, an increment can then be added to the absolute minimum coefficient acceptable at the particular test speed to establish the minimum corresponding to the mode of measurement used. For example, if it is known that locked-wheel testing is the mode of measurement which gives the lowest coefficient on a particular type of surface at 50 km/hr, and if the minimum value prescribed by international standards is 0.4 at this speed, then a highway department which uses 17 percent brake slip should add an increment (perhaps 0.05, this being known from the correlation plots suggested above) to this minimum value. The minimum value which the highway department would recognize when measured with their 17 percent brake slip tester, would then be 0.45.

DISCUSSION AND CONCLUSIONS

The reduction of all existing international standards to one mode of testing based on correlation equations as described above would provide a much more precise set of data than that presented in the Figure 1, and it is certain that the agreement in minimum values for skid resistance would be closer. The reduction procedure would ultimately apply to the nature of road surfaces (asphaltic or concrete) and type of test tire, but in terms of speed dependence at least two distinct test speeds should be preserved. An alternative to this procedure is to specify one speed of testing combined with a prediction of skid-resistance gradient from pavement geometry (11).

The Permanent International Association of Road Congresses has already specified the value of 0.4 at 50 km/hr as the absolute minimum acceptable coefficient of friction obtained with any test mode. The specification of another minimum value at a higher test speed (perhaps 80 or 90 km/hr) would provide an acceptable international standard, and it is hoped that this will be accomplished in the near future. A recent survey of skid-resistance requirements for main rural highways in the United States (12) has suggested somewhat different minimum values of skid-resistance than those proposed by the Permanent International Association of Road Congresses, and the authors have attempted to reduce their recommended values to one speed of testing. While this contribution is worthwhile, it is suggested that the existing international standard of 0.4 at 50 km/hr be adhered to until further refinements are introduced, and in the meantime it can, of course, be extended to include a higher test speed. Further work should therefore concentrate on the development of correlation equations of the type of Eqs. 1 through 4 in this paper, so that the new international skid-resistance standard can be interpreted in terms of local testing modes.

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The Logical Design of Optimum Skid-Resistant Surfaces

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A logical sequence is suggested for the design of optimum skid-resistant pavements. The mean wavelength and slope of the texture may be determined from drainage requirements at the average maximum speed for the section of pavement under consideration, and the values may be checked by the demands of the hysteresis contribution to friction in the skidding mode. For surfaces where the asperities are rounded by wear and the demands of traffic, it is necessary to provide a micro-roughness at asperity peaks to establish adhesion between tire and road in wet rolling. The amplitude of the micro-roughness must be greater than the elastohydrodynamic water-film thickness which would otherwise exist at asperity tips due to relative slip between tread and road in driving or braked rolling, whereas the adequacy of its sharpness is indicated qualitatively by "feel." It is estimated that for practical road surfaces the wavelength lies in the range 3 to 20 mm, and the micro-roughness has an order to magnitude of at least 10 to 100 microns.

•IT IS a well-known fact that the coefficient of sliding friction on a wet road surface decreases with increase of speed (1), and that the rate of decay is a function primarily of drainage ability. Thus, coarser surfaces having inherently greater void volume between individual asperities (and hence greater drainage capacity) exhibit a lesser rate of decay than finer surfaces. The fallacy of attempting to characterize the skid resistance of a particular pavement by establishing one friction value at the speed of testing is at once obvious. Two pavements (one fine and one coarse) may have the same coefficient of sliding friction at a particular sliding speed, but since the slopes of the coefficient of friction vs velocity curves are entirely different, the finer texture is superior at speeds below the test speed and the coarser texture is superior at speeds exceeding the latter.

The situation is not serious if the test speed corresponds either to the speed limit of the particular pavement section, or to the average maximum speed (in cases where no speed limits are posted), since the pavement is then rated at its worst friction value. However, if the test speed is considerably less than the speed limit (or average maximum speed) as shown in Figure 1, it is clear that the measurement of friction values at one speed is not sufficient. Either one or more additional speeds (clearly different from the original test speed) must be selected at which to measure skid resistance, or otherwise the gradient of the friction vs velocity curve for that pavement must be established from drainage considerations. The latter information combined with the measurement of skid resistance at one sliding speed is sufficient to determine frictional performance over a range of speeds.

OPTIMUM SURFACE TEXTURE

The mean wavelength and slope of the individual asperities of the road profile can be shown to give a maximum hysteresis contribution to friction at a particular sliding speed (2). For typical road surfaces and rubber materials, this speed normally ex-

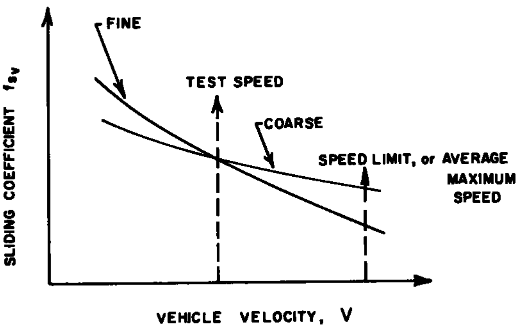

Figure 1. Friction/velocity curves on fine and coarse surfaces.

ceeds the average maximum speed (even in Europe). It is also necessary that the mean wavelength provides asperities which are sufficiently large to insure adequate drainage of water into the neighboring voids (3) at some typical maximum speed. In practice, the mean wavelength selected may be a compromise between drainage considerations (which suggest a lower size limit) and hysteresis requirements (which indicate an upper limit) at the speed limit or average maximum speed for the pavement under consideration. However, since the hysteresis requirement pertains to the sliding mode alone and drainage considerations apply to both rolling and sliding, it is certain that the drainage criterion predominates in the final selection of wavelength.

The adhesion contribution to friction can be maximized by providing in addition a sufficiently sharp texture. Figure 2a shows one form of an idealized random road surface, where the individual asperities are pointed to provide sufficiently high localized pressures between tread and surface, which will break through water films entrained by elastohydrodynamic action (4) as the wetted rubber slips over the road asperities. Sandpaper surfaces largely exhibit this profile, although the scale of the wavelength is much too small to match the hysteresis/drainage requirements of road surfaces.

Actual road surfaces have profiles more in accordance with that shown in Figure 2b. Here again, the hysteresis and drainage conditions may be satisfied by choosing the wavelength in accordance with a maximum design skid-speed, but it is clear that since the asperity tips are predominantly rounded rather than pointed, it will be necessary to carefully select an adequate micro-roughness at asperity peaks to eliminate skidding according to a method previously outlined (5).

The selection of optimum surface texture follows a rational procedure which is depicted in Figure 3. The mean wavelength and slope are dictated by hysteresis/drainage factors, whereas the micro-roughness is selected to insure the existence of an adhesional mechanism at asperity peaks in defiance of water-film entrainment. The various factors involved are grouped for convenience in the following categories:

1. Driving conditions (forward speed, rolling/sliding, nominal slip and wet/dry);
2. Tire properties (viscoelasticity, tread design);
3. Interaction events (hysteresis, drainage, elastohydrodynamic factors, hydroplaning); and
4. Pavement geometry (wavelength/mean slope, micro-roughness).

ELASTOHYDRODYNAMIC CONSIDERATIONS

Perhaps the most significant interaction event occurring when a pneumatic tire rolls with brake or drive slip on a wet road surface, is the generation of hydrodynamic

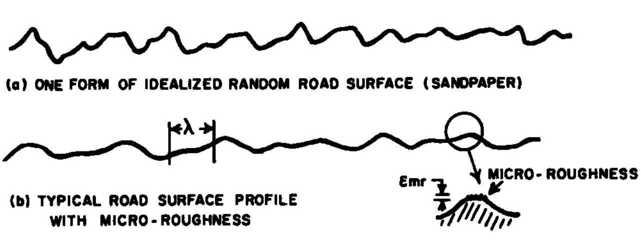

Figure 2. Idealized and typical road surfaces.

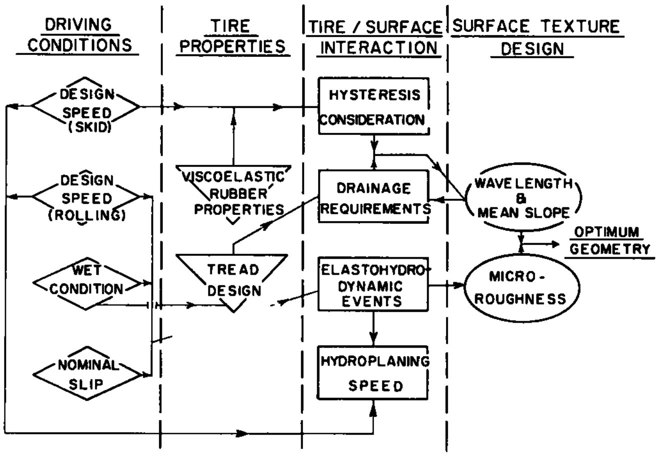

Figure 3. Sequence of events in selecting optimum surface geometry.

pressure wedges on the leading edges of road asperities, as a result of relative slip between tire and road in the contact patch. The degree to which the rubber is deformed outward from the road profile by such pressure generation is determined by tread viscoelasticity and inflation pressure (4, 5), and there must be a compatibility between the hydrodynamic and viscoelastic pressure distributions (Fig. 4). The resulting elastohydrodynamic or viscoelastohydrodynamic pressure acting at asperity tips in the road profile can then be used to determine the minimum film thickness from lubrication theory, and the amplitude of the minimum micro-roughness ϵ_{MR} required at asperity tips to eliminate or counteract this effect is thereby prescribed. Figure 5 shows part of the rear of the contact area in wet rolling at any instant, where the generation of hydrodynamic pressures on individual asperities of the road surface attempts to entrain water over the tips of the latter and thereby to separate tread and road locally.

In theory, however, not all road surfaces require the existence of a micro-roughness (Fig. 6). Let it be assumed that the wavelength λ has been selected previously by hysteresis/drainage requirements, but that the mean slope is variable within prescribed limits. It is also stipulated that increasing slope and sharpness are related in some manner. The peak pressure on any asperity increases rapidly and nonlinearly with mean slope to a critical value at the point C, but the elastohydrodynamic pressure distribution can be sustained only in the region A to C. As the sharpness increases be-

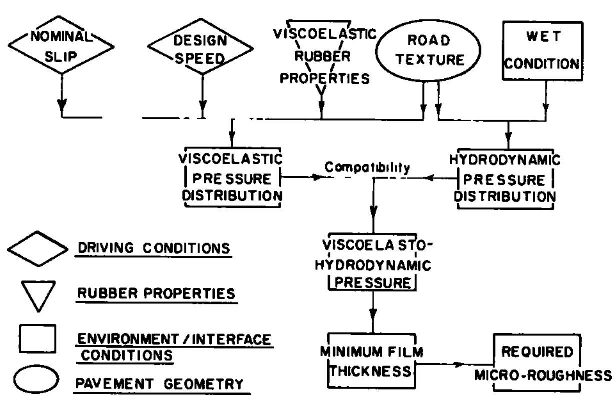

Figure 4. Selection of an optimum micro-roughness for a given pavement.

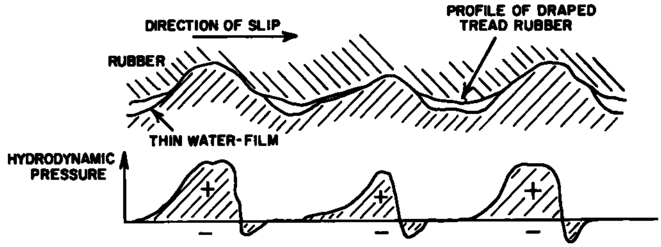

Figure 5. Generation of hydrodynamic pressures on individual asperities of wet road.

yond the value at C, the corresponding elastic pressure is too great to permit the existence of a water film at asperity tips, and thus the hydrodynamic pressure component has zero value along DE. The elastic or viscoelastic pressure distribution continues to increase indefinitely along CB.

It is apparent that in the region C to B no micro-roughness is required to counteract water-film entrainment, since none exists in this range. This type of surface is similar to the sandpaper profile depicted in Figure 2a, although the wavelength would, of course, be greater. For very small slopes, there will be inadequate drainage in the range A to F. The design limits for the selection of micro-roughness are therefore clearly specified by C and F (F^1) as indicated in Figure 6. The extent of the range CF^1 remains to be determined from further research, but it is certain (4, 5) that λ has an order of magnitude of 3 to 10 mm and ϵ_{MR} ranges from 10 to 100 μ.

FLOODED AND DAMP CONDITIONS

In flooded conditions, the tread grooving of the tire and the mean void width of the pavement must be capable of discharging an adequate amount of water from the contact patch at the average maximum speed for that particular road section. Other investigators (2) have shown that when a rolling tire is braked on a flooded road surface, the available coefficient of friction falls off much more rapidly with increase of speed beyond the design limit for that pavement (Fig. 6). Furthermore, as the design limit is increased (by appropriately increasing the wavelength and drainage capacity of pavements), there is a loss of braking capacity at speeds below the average maximum speed or design limit. Thus, it is clear (Fig. 6) that pavement B is designed for a higher speed range than A, but the performance of B is inferior to that of A below the

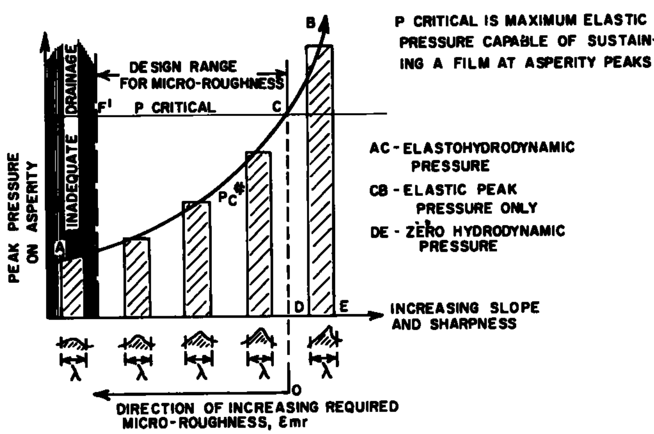

Figure 6. Micro-roughness requirements for different asperity shapes.

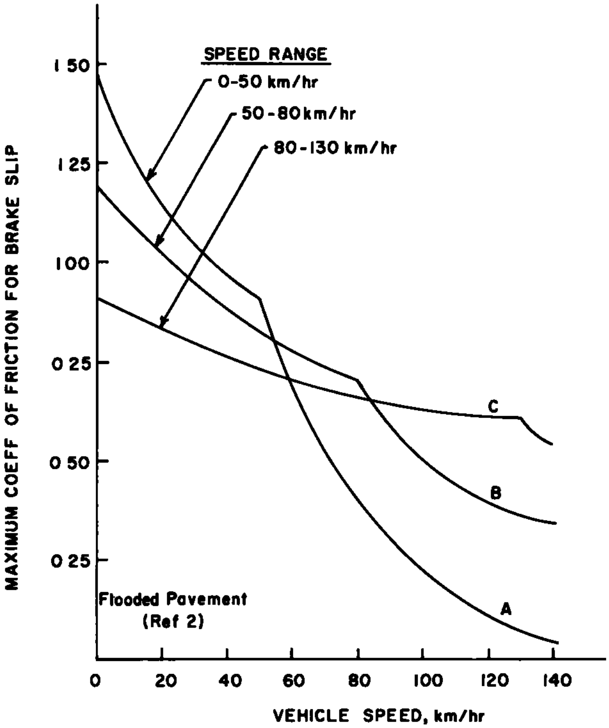

Figure 7. Optimum surface roughness for flooded pavements (2).

design speed limit of the former. A similar argument applies to surface C, when compared with either A or B. In practice, it is desirable to have as constant a friction coefficient as possible over the design range of speeds for any pavement. Thus, the size of wavelength which has already been tentatively selected as a compromise between hysteresis and drainage considerations must lie between a minimum value (determined from the requirement that the transition point, which is apparent in Figure 7, lies outside the speed range) and a maximum suggested by the sacrifice in braking ability at lower speeds within that range.

An interesting application of these principles is the optimum design of runway surface texture. If it be assumed that aircraft may land in either direction on a particular runway, then it is clear that maximum rolling velocities (equivalent to touchdown speed) occur at either end of the landing strip. Now the deformation frequency upon which the hysteresis contribution to friction depends is given by the ratio of forward speed to pavement wavelength. It is therefore apparent that the wavelength selected to maximize skidding friction is greatest at touchdown (or take-off) speeds, which corresponds to the ends of the runway. This criterion also satisfies drainage requirements. As the aircraft speed is reduced after initial touchdown, the wavelength of the texture will decrease gradually towards the center of the runway length. The optimum surface texture will therefore exhibit a decreasing wavelength as the center of the runway is approached from either end. The question arises: Why not preserve the same texture which is presumed adequate for touchdown speeds, since drainage requirements are more than satisfied at all lesser speeds? However, Figure 7 shows that unless the wavelength matches speeds at all sections, there may be a loss in braking effectiveness on flooded landing strips. This, of course, would increase the length of runway required for safe braking under wet conditions.

With damp pavements, the phenomenon of viscous hydroplaning (4) may occur. This is due to the entrainment of a very thin water film over asperity tips in the road texture, as a result of localized slip between tread rubber and road surface in rolling. Whereas the mean wavelength and slope of texture is designed for the flooded condition from hysteresis/drainage requirements, the criterion for the selection of an adequate micro-roughness at asperity peaks is the effective counteraction of elastohydrodynamic film entrainment under damp or thin film conditions. The micro-roughness permits effective adhesion to take place between rubber and surface even in the presence of thin films.

Although the flooded condition is relatively rare on roads and runways, the damp or thin-film situation occurs whenever there is any precipitation whatsoever or even a high humidity. Furthermore, the viscous hydroplaning phenomenon occurs at the rear of the contact patch in wet rolling, when the front part experiences dynamic hydroplaning under flooded conditions. The adhesion-generating mechanism is therefore the principal contributor (4) to braking effectiveness in wet rolling, irrespective of the degree of precipitation or film-thickness.

The gross relative slipping velocity between tire and surface in the rear of the contact patch for wet rolling necessitates the existence of micro-roughness at asperity peaks to maintain effective adhesion and oppose the lubricating effect. At the same time, the rubber elements in the front of the contact patch (for thin-film conditions) have virtually no longitudinal motion relative to the road surface. Here, the sharpness of the micro-roughness itself must be capable of discharging minute droplets of the film into neighboring voids. It should be noted that the distance moved by these droplets is infinitesimal, since this is consistent with the observation that very thin films when impacted by tread rubber in high-speed rolling must behave like solids (6, 7) in transmitting high shear forces with no finite displacement. The effectiveness of the micro-roughness in permitting this phenomenon to take place is still best described by Giles qualitative test (8)—the "feel" of the surface texture. Surfaces which are sufficiently harsh to the touch may therefore be deemed to have an adequate sharpness of micro-roughness; at the same time, the minimum permissible amplitude of micro-roughness is determined by elastohydrodynamic considerations in the rear of the contact patch as described earlier.

CONCLUSIONS

A logical design sequence for the selection of an optimum surface texture in roads and runways has been proposed on the basis of research performed by the author and other investigators. It is concluded that the mean wavelength and slope of texture is chosen from drainage requirements (with consideration of the contribution of hysteresis to skidding friction) at the average maximum speed or design speed limit for the particular pavement under consideration. For surfaces which are sufficiently pointed and sharp (Figs. 2a and 6), there is no need for a micro-roughness at asperity tips, since the elastic pressure peak is sufficiently great to preclude the existence of a continuous water film. Most road surfaces have rounded asperities, however, and it is necessary to design a micro-roughness to oppose the lubricant effect at asperity peaks due to fluid entrainment. The amplitude of the micro-roughness should exceed the elastohydrodynamic film thickness which would otherwise exist at asperity peaks, and its sharpness should permit the displacement of fluid droplets through an infinitesimal distance to establish adhesion between tire and surface. It has been shown (4) that adhesion contributes substantially to the coefficient of friction in wet rolling.

It is certain that the wavelength for road surfaces lies in the range 3 to 30 mm (depending on average maximum speed), whereas the micro-roughness may be of the order 10 to 100 μ. The sharpness of micro-roughness is qualitatively measured by its feel, but no mechanical measure of this parameter has yet been proposed. Considerable work has been done on the drainage of road surfaces and in the prediction of skid-resistance gradient (3, 8, 9) and in the general evaluation of surface texture as related to its friction-generating potential (10, 11). Yet there is need for further refinements in establishing the exact geometry of the optimum surface texture for a given set of environmental conditions.

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HARTWIG W. KUMMER

Hartwig W. Kummer died on August 8, 1968, at the Sloan-Kettering Memorial Hospital in New York. "Pete" was born in Adenau, Germany, on January, 6, 1929. He received his Dipl-Ing. from the Technical University Hannover (Germany) in 1957, and came to the United States in the same year. This was just around the time when the Automotive Safety Research Program got under way and he became its first full-time research assistant. His first task was the design of the skid-trailer, from which eventually the Penn State Road Friction Tester evolved. He supervised the first skid resistance survey of the Pennsylvania Highway system in 1961.

He wrote and published numerous papers in a variety of publications, from the Highway Research Records to the Transactions of the Rubber Division of the American Chemical Society. In the process, he also obtained a Ph.D. degree in Mechanical Engineering at the Pennsylvania State University in 1965 and became a licensed Professional Engineer in Pennsylvania in 1966.

Five days after his death, he was scheduled to deliver two papers at the Highway Research Board Summer Meeting in Denver.

At the time of his death, he was a member of ASTM Committee E17 on Skid Resistance and chairman of its subcommittee on fundamentals. He was also active on Highway Research Board Committee D-B4 on Pavement-Vehicle Interaction.

Pavement Friction and Temperature Effects

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The magnitude of friction produced by two bodies in rubbing contact is, among other factors, determined by their material properties. Whenever these properties change, friction will change also. Since rubber is a viscoelastic material, whose elastic and damping properties are strongly affected by temperature, the friction of rubber sliders or skidding tires is likewise affected by temperature.

To better understand the effect that temperature has on pavement friction, the adhesion and hysteresis components are separated and their temperature dependence is studied independently. Whereas the adhesion component may increase or decrease with temperature, depending upon sliding speed, the hysteresis component is usually reduced by temperature. By superposition of the adhesion and hysteresis curves, the temperature dependence of friction can be qualitatively predicted.

Field and laboratory tests made with skid trailers and portable testers confirm this temperature dependence. The experimental results are often difficult to interpret or sometimes ambiguous, however, because the data are influenced by factors other than temperature and reflect the sum of adhesion and hysteresis, both of which are temperature dependent in a different way.

For these reasons correction factors are difficult to obtain ant at present none are available which would permit normalization of friction measurements to a specified temperature within known confidence limits.

•THAT there is some relation between temperature and pavement friction has been known for some time. When Giles and Sabey (1) related the mean monthly air temperatures to the percentage of the total accidents in which skidding on wet pavements occurred, they found that this percentage was changing seasonally and closely paralleling the mean monthly temperatures (Fig. 1). One cannot deduce from these data, however, that temperature is the only factor causing the change in the incidence of skidding accidents. Indeed other data (Fig. 2) show that the frequency of skidding accidents is greater in fall than in spring even though, in first approximation, the mean temperatures in spring and fall should be alike (2, p. 38).

The seasons not only differ in temperature, but there may be other seasonally induced effects which influence accident frequency. Pavement surfaces may change, average tire conditions may differ, driver response to pavement slipperiness may be conditioned by road and weather conditions, etc. To learn whether temperature affects accident frequency via changes in skid resistance a look at how the latter changes during a single day should be informative.

Figure 3 gives the data for a single 24-hr period (3). The skid resistance is higher when the temperature is lower, and vice versa. Although in this case the maxima and

*Deceased.

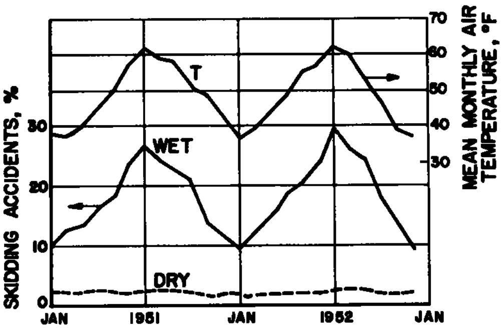

Figure 1. Seasonal variations of the percentage of skidding accidents and of the mean monthly air temperatures in Great Britain (1).

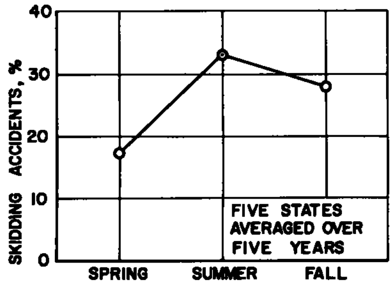

Figure 2. Seasonal variations of the percentage of wet skidding accidents in five states (2).

minima do not exactly coincide there is no doubt that some relation exists between temperature and skid resistance. This is even better illustrated by Figure 4 (4).

Obviously then, in describing the frictional characteristics of pavements, temperature must be taken into account. Since it is not practical to postulate that field tests are to be made at a single temperature, a method for correcting the obtained data to a standard temperature would be extremely helpful if precise comparisons between pavements are to be made. On the other hand, one should also be able to assess the effect of temperature on the frictional performance of commercial tires if reasonable traffic rules and practices are to be postulated or if the accident potential of pavements is to be predicted.

RUBBER FRICTION AS FUNCTION OF TEMPERATURE

In the frictional interplay between wet pavement surface and tire it is primarily the tire which changes characteristics with temperature. In this paper we address ourselves to the problem of how changing tire or rubber temperature affect friction. This is not to say that pavement and water temperatures may be ignored, but they are mostly effective through the manner in which they influence the rubber temperature at the interface.

Wet rubber friction has two principal components: that caused by adhesion (surface friction) and that caused by hysteresis (internal friction), see Figure 5. On any road surface both components are generated, though their relative magnitudes change with the character of the surface. Unless both components respond in the same manner to temperature changes, the friction-temperature relationship will not be the same on

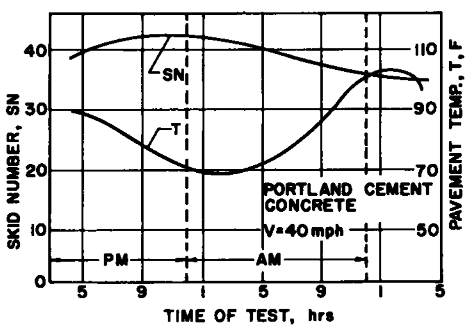

Figure 3. Daily variation of skid resistance and temperature (3): locked-wheel tests with road friction tester.

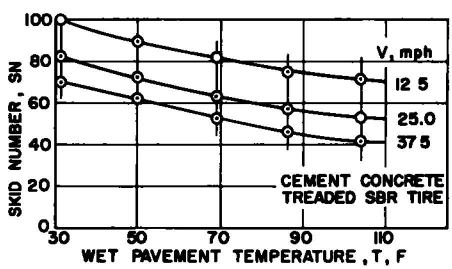

Figure 4. Change of skid resistance with temperature at several speeds: locked-wheel tests with road friction tester.

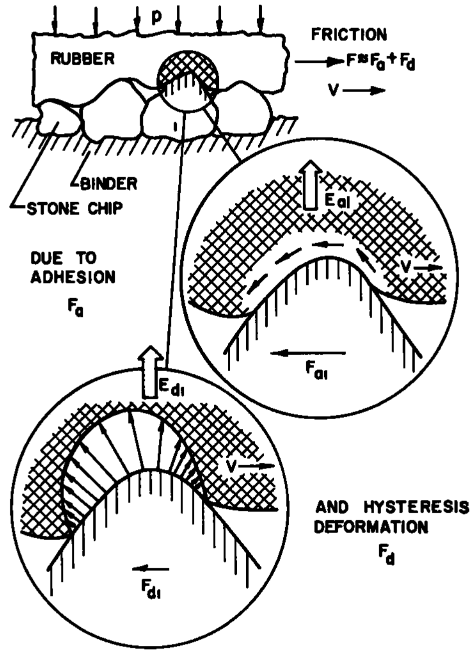

Figure 5. Friction has two principal components: adhesion and hysteresis.

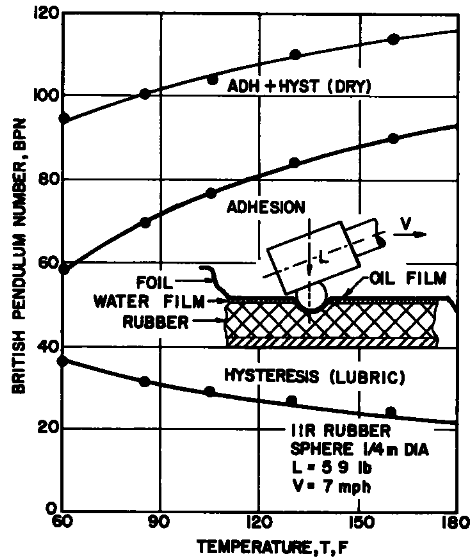

Figure 6. Separation of the friction components by lubricated foil technique (5).

all pavements even though tire or rubber are the same. This mechanism explains at least partially why conflicting data on the effect of temperature are being obtained.

In the laboratory it is possible to separate the two friction components (5). The upper curve of Figure 6 was obtained by sliding a polished steel ball over a dry rubber specimen (the British pendulum tester was modified for this experiment). The lower curve was obtained by placing a thin plastic foil on the rubber and lubricating the foil with a light lubricant. This effectively suppressed most of the adhesion. The difference between the friction values for the two experiments then represents the adhesion component. On wet pavements the adhesion component is of course proportionally much smaller than shown here.

Of interest here is the fact that the adhesion and hysteresis components have different temperature responses, both in sign and in magnitude. The net effect in the present case is a positive temperature-friction gradient. This is in direct opposition to what Figures 3 and 4 show. It must be borne in mind, however, that Figure 6 applies to (a) a particular rubber compound, (b) a particular sliding speed (which is much lower in the case of Fig. 6 than for Figs. 3 and 4), and (c) a particular type of contact (a single steel ball at some arbitrary load vs. the contact patch of a sliding tire on a pavement).

In Figure 7, several rubber compounds were used in the same type of experiment, the plotted results representing total friction in the absence of lubrication. The curves illustrate that not only peak friction values vary, but also that the peak values occur at different temperatures. At a given temperature the friction-temperature gradient for one rubber compound can be positive, while for another it is negative or zero. This means that the relative ranking of the compounds at one temperature is not necessarily the same as at another.

Figure 7 also shows that if one is free to choose the compound and the temperature one can, for a given experiment, achieve insensitivity to small temperature variations. This can be used to improve the precision of routine data acquisition programs when precise temperature control or measurement is impractical. It is, however, necessary to verify the insensitivity to temperature variations over the entire anticipated operating spectrum.

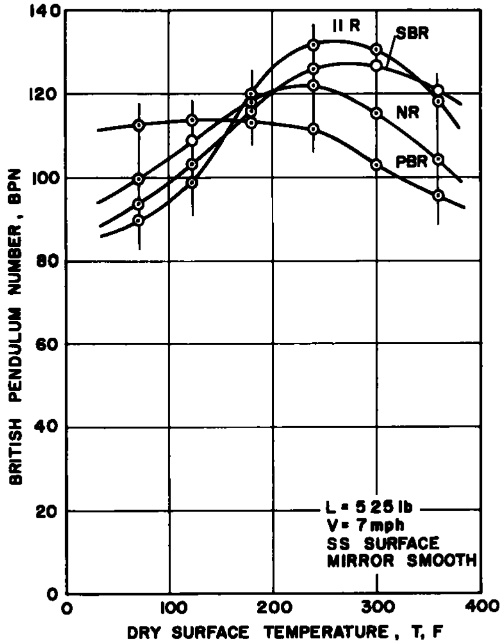

Figure 7. Temperature sensitivity of the friction of four rubber compounds: total dry friction under conditions similar to those of Figure 6.

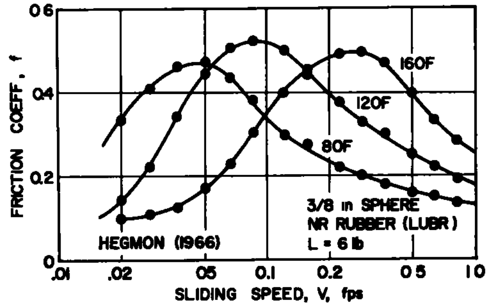

Figure 8. Coefficient of friction as function of low sliding speed at three different temperatures.

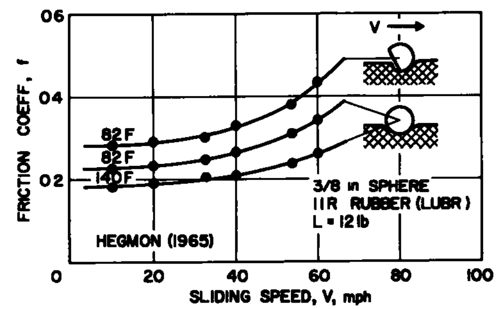

Figure 9. Coefficient of friction at high sliding speed and two temperatures.

For the tests of Figures 6 and 7, the sliding speed was constant. If it is varied, the friction peak will occur at a different temperature. Conversely, if temperature is varied, the friction peak occurs at a different sliding speed (Fig. 8). The data again represent the results obtained with a sliding polished steel ball, but this time in the presence of a lubricant (6, p. 23-25). It should be noted that the sliding speeds for Figure 8 are quite low—even the 160 F peak occurs at only 0.3 fps or 0.2 mph.

As sliding speed is increased friction decreases, but eventually increases again (Fig. 9). The experiments from which the data are taken (7) could not be carried to high enough speeds to reach the second peak. Covering the entire speed range in one continuous experiment results in a curve of the type shown in the upper graph of Figure 10. The solid curve represents the total observed friction, whereas the broken line is that due to hysteresis only, as determined by means of a refined version (6, p. 63-69) of the foil method. The peak at the low sliding speed is almost entirely the result of a maximum of the adhesion component. The high speed peak is caused by the peaking of the hysteresis component since, at least in this case, the adhesion component has completely disappeared (the smooth sphere hydroplanes). It is therefore appropriate to speak of an adhesion and a hysteresis peak, respectively.

As already pointed out, the adhesion peak occurs at very low speeds. A sliding tire always operates to the right of it, but the fact that the peak moves with temperature does concern us here. This is brought out by the lower graph of Figure 10; the normal operating speeds for three types of skid-resistance measuring instruments are shown in relation to friction curves for four different temperatures. From Figure 8 we know that the adhesion peak moves to the right with increasing temperature; consequently T_1, T_2. . ., designate curves for progressively higher temperatures. It can be seen that at ST, the standard speed for skid tests with a road friction tester of the locking wheel type, increased temperature will cause a decreased coefficient of friction to be

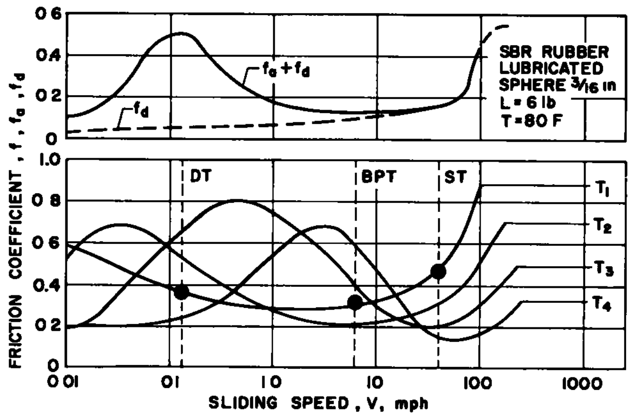

Figure 10. Coefficient of friction over a wide range of sliding speeds—(top) constant temperature, f_a = adhesion coefficient, f_d = deformation (hysteresis) coefficient; (bottom) four temperatures, T_1, T_2,.... DT = Penn State drag tester, BPT = British pendulum tester, ST = skid trailer.

measured. The same is not true, however, if the British pendulum tester (BPT) were used. Here the coefficient would be at a minimum at T_2. Using the Penn State drag tester at very low speed (DT) would place the minimum at T_4, the highest temperature shown.

This example shows why no generalized statement about the temperature-friction relationship can be made even if all variables, except temperature and sliding speed, remain constant. The horizontal shift of the friction vs. speed curves shown in Figures 8 and 10 have been used by Grosch to show that by application of a suitable transform they can be combined into a single master curve. The concept permits substituting a sliding speed change for a temperature change and vice versa. Thus, with the generalized shape of the friction vs. speed curve in mind (top of Fig. 10), it is not difficult to analyze, at least qualitatively, the causes of observed changes of friction with temperature. In practice, obtaining or using such a master curve may encounter certain difficulties because of the superimposition of hydrodynamic effects, problems of measuring or controlling temperatures, self-heating of the rubber at high sliding speeds, etc.

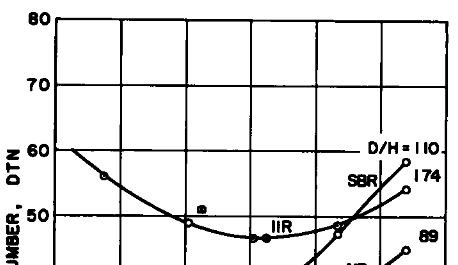

Figure 11. The temperature sensitivity of drag tester sliders: averages of results on several different surfaces—D/H = damping/hardness ratio (Damping determined by rebound method ASTM D 1054, Hardness acc. to ASTM D2240).

EXPERIMENTAL EVIDENCE AND ITS INTERPRETATION

In Figure 11, the results of tests performed with the Penn State drag tester are shown. The experiments were carried out to determine the effect of changing the rubber compounds of the slider. The sliding speed (2.35 ips or 0.13 mph) had been selected to obtain minimum sensitivity with SBR rubber over the temperature range normally encountered in the laboratory. It can be seen that the DTN (drag tester number) is virtually constant between 60 and

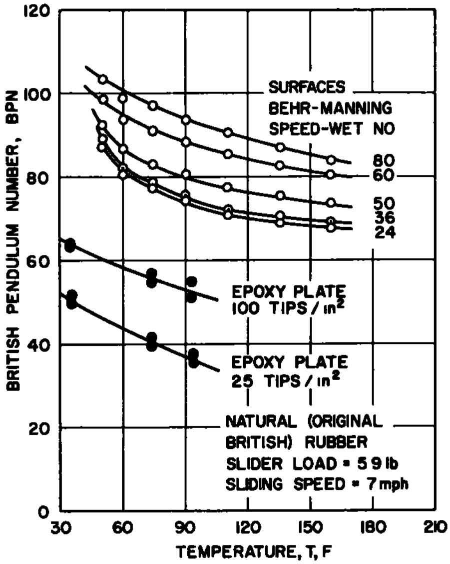

Figure 12. The combined effect of temperature and surface characteristics on friction measured with the British pendulum tester.

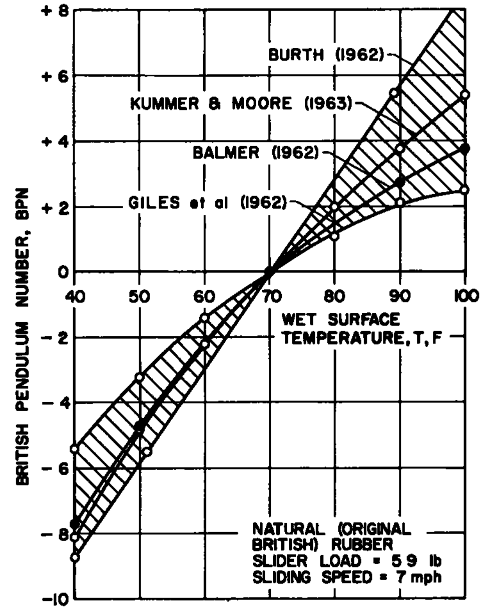

Figure 13. Corrections derived from several sources to normalize British pendulum numbers to 70 F.

80 F. With different compounds the minima (solid dots in Fig. 11), and thus the flat portions of the curves, occur at other temperatures. If the experiments were to have been designed around the use of a compound other than SBR the sliding speed would have had to be lowered in order to move the minimum to 70 F.

This is a practical application of the concepts discussed earlier, but it may be confusing that in the case of Figure 11 minima should occur when earlier only maxima were considered. The occurrence of minima can, however, be explained by reference to Figure 10. As a result of higher temperature, the adhesion component of the friction increases in the region of interest and the hysteresis component decreases. At first the decrease exceeds the increase, so that there is a net decrease in observed friction. With a further temperature increase a point will be reached at which both changes cancel each other: the minimum point of the total friction curve is reached. Further heating will again result in a net rise. (The BPT line in Fig. 10 illustrates such a situation: the coefficient is higher than at T_2 whenever the temperature is either higher or lower than T_2.)

For the curves of Figure 11, the results from six different surfaces have been averaged. Figure 12 shows how different surfaces influence the temperature sensitivity. No minima were reached in this case because the sliding speed was higher than for Figure 11. The shape and the number of asperities per unit area influence not only the general friction level, but also the temperature sensitivity. Whether the latter effect is significant or not cannot be stated generally, if only because not enough data are available and because different applications involve the rubber differently and the range of surface characteristics varies from application to application.

TEMPERATURE CORRECTIONS

If one would attempt to provide a temperature correction to data obtained with the British pendulum tester this does not seem too difficult a task at first. That it is not a simple problem is illustrated by Figure 13. Data reported by several authors have been plotted in terms of the BPN (British pendulum number) which must be added or subtracted to correct the observed BPN to 70 F. At temperatures below 70 F the different sources agree reasonably well, but above 70 F there is considerable spread. Not enough information is available to rule out the possibility that a good part of the spread comes from differences in experimental technique. Another factor is undoubtedly that different surfaces were used for each set of data.

Burth (8) used cement concrete, Kummer and Moore (9) abrasive paper (the raw data are those shown in Fig. 12), Balmer (10) machined epoxy (see also Fig. 12), and Giles et al (11) eight different road surfaces. Precisely what characteristics of the surface must be considered in a correction formula cannot be deduced from the information given in these sources. One might favor the correction suggested by the Giles et al data because they come from actual road surfaces, but before making a choice one would have to know why Burth's data from cement concrete surfaces fall on the opposite side of the range shown in Figure 13.

These complexities are illuminated, though not resolved, if the friction process through which the slider of the British pendulum tester goes is investigated in more detail. In Figure 14, the friction history of three different passes is shown. They differ from the standard pass in that the sliding length is somewhat greater than normal and that the slider was forced to move at constant speed. The dependent variable is therefore not the total energy loss, but the instantaneous coefficient of friction. (It was measured by supporting the test specimen on an air bearing and biasing the specimen against a pressure transducer with a very high spring rate.) The coefficient rises rapidly to a maximum, which corresponds to the adhesion peak of the friction speed curve, and then drops off gradually as the slider edge heats up. Since the test surface was extremely smooth stainless steel, there is little difference between the dry and wet condition. The slider wipes away the water almost completely. Therefore, the friction in this case is almost entirely due to adhesion. When a wetting agent is added to the water the adhesion component is suppressed and only hydrodynamic, viscous and interfacial tension forces remain; even their sum is almost negligible under the conditions of the experiment.

If a less smooth surface had been used the process would have become still more complex. It is therefore not difficult to appreciate that surface characteristics can significantly affect the manner in which temperature influences friction as measured with a pendulum device. According to Figure 14, the initial temperature of the rubber slider should have little influence on the integrated coefficient of friction, but this can be said with certainty only about nearly perfectly smooth surfaces.

It is not surprising that locked-wheel tests with full-scale tires give even less agreement on how to correct for temperature (Fig. 15). Only the Kummer and White data were obtained with the ASTM standard test tire. The rest of the tests employed differing tires and test speeds.

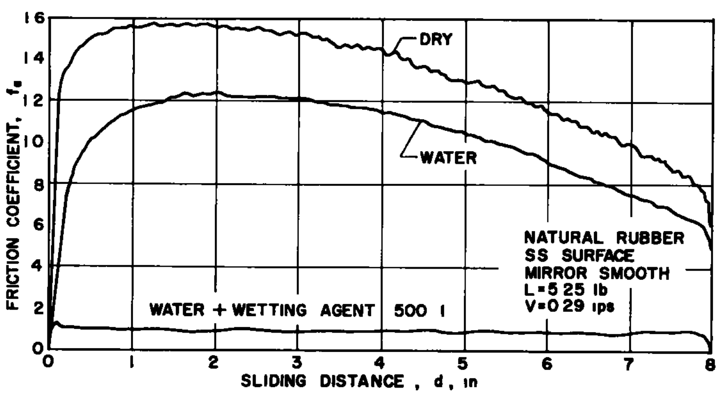

Figure 14. Instantaneous coefficients of friction when the slider from a British pendulum tester passes across a very smooth surface at constant speed.

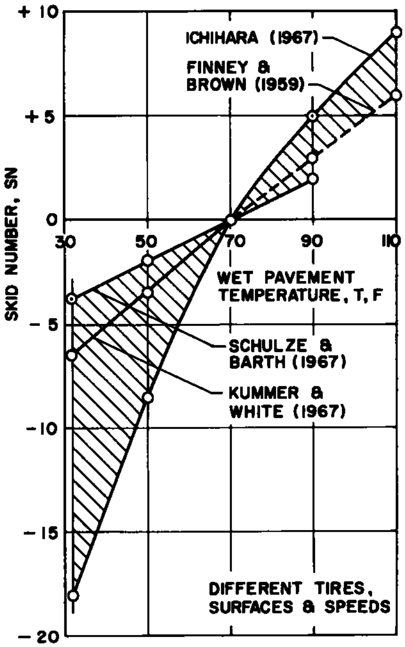

Figure 15. Corrections derived from several sources to normalize skid numbers to 70 F: locked-wheel tests of different tire types at different speeds on unidentified surfaces.

CONCLUSIONS

1. The friction-temperature gradients are in practice always negative for the currently used skid testers. The magnitude of the gradients is, however, still quite uncertain even for the ASTM standard tire and the pendulum tester slider made of either ASTM rubber or British natural rubber.

2. Since different surfaces cause differences in the temperature gradients, compounds which, in the operating range, are least temperature sensitive have advantages. Small gradients result in smaller errors if the surface characteristics are not or cannot be taken into account or if the temperature measurements are not precise.

3. Friction-temperature gradients are a function of surface characteristics because the varying contributions of the adhesion and hysteresi components to the total friction differ. Because the two components have different temperature characteristics the effect of temperature changes is so complex that the effect probably can never be defined quantitatively in a rigorous way except statistically on the basis of a large number of carefully controlled experiments.

4. Although the temperature of the rubber is responsible for the observed temperature dependence of tire or slider-pavement friction the temperature of the pavement and of the water used for wetting it do play a part because of heat transfer across the contact area. In routine tests it is, however, impractical to measure more than one temperature. Without extensive experimentation it cannot be stated how and where this temperature should be measured. Any correction using it would contain a degree of uncertainty. Experiments would have to define the limits of the possible error. The error might be reduced by more rigid test procedures than are now being used.

5. When compliance with a standard must be shown and the observed values are close to the cutoff value it may be necessary to make the compliance tests while the ambient temperature is within specified limits. In conjunction with a tightly controlled test procedure this would eliminate the uncertainties which arise from the complex effects caused by temperature changes.

ACKNOWLEDGMENTS

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This paper was to have been written by H. W. Kummer. He had prepared notes for it, before death put a too early end to his career. The text before the reader was written by W. E. Meyer, with assistance from M. O. Schrock in the interpretation of the Kummer notes.

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Relation Between Wear and Physical Properties of Roadstones

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One aspect in the skid-resistant life of a pavement is the polishing of individual roadstones by abrasives on the road. The related problem studied here is the gradual wear (microscopic scale) of homogeneous roadstones to determine the pertinent physical properties of these minerals in the wear process. Wear is measured as a weight loss of material. A brief review of wear is given. A wide range of concepts exists, most studies pertaining to metals. Some of the important parameters are melting temperature, hardness, elastic modulus, and energy.

Ten mineral samples, predominately oxides, were held against the rubber tracks of a rotating drum in the presence of dry fine abrasives. Three loads and speeds were tested for each of three different abrasives. These test conditions simulated actual pavement experience. Microscopic photographs of worn surfaces revealed two phenomena: scratching and pitting. Rapidly wearing minerals suffered both types of damage while slow wearing minerals displayed no scratching, only a small amount of pitting. Rapid wear occurred when the abrasive was harder than the mineral. Wear was proportional to load and inversely proportional to hardness. The slow wear of minerals softer than the abrasive was independent of load. Limited evidence suggested that this wear depended on the ratio of surface to strain energy in a given mineral. Pitting was related to energy concepts while scratching was related to hardness. No correlation could be achieved between wear and melting temperature or modulus. Hardness was found to be the most important parameter in the rapid wear of homogeneous minerals.

•THE predominant variable in tire-pavement friction during wet conditions is the surface texture of the pavement itself. Skid resistance depends on a surface which provides film penetration and drainage channels. One of the factors which can alter these characteristics is the smoothing and rounding of exposed roadstones by abrasives found on the road. This process is caused by particle removal, or wear, on a microscopic scale, and it is commonly referred to as polishing. The amount of this wear over a period of time can have a direct effect on the skid-resistant life of a pavement.

The problem of concern here is the gradual wear (on a microscopic scale) of homogeneous roadstones which occur as constituents in the heterogeneous aggregate. Specifically, it is the nature of the particle removal from the stone surface. A weight or volume loss of material will be used to define the rate of wear. Attempts will be made to (a) relate the wear to physical properties of material and (b) define the probable wear mechanism. To this extent it would be desirable to duplicate as closely as possible the conditions found on the road.

NOMENCLATURE

A_a = apparent contact area;
A_o = equivalent area of piled-up material on a groove;
A_r = real contact area;
A_v = area of the vertical face of a groove;

c_n = any constant;
d = diameter of hemispherical wear particle;
E = elastic modulus;
F = friction force;
f = coefficient of friction;
H_v = Vicker's hardness;
P_m = material flow pressure;
S = surface energe of formed wear particle;
σ_{yp} = material yield stress;
V = sliding velocity;
v = volume of wear removal for a given distance of travel;
W = load; and
W_o = component of wear removal independent of load.

WEAR CONCEPTS

The complexity of wear processes has led to a large diversity in both theory and approach to its problems. This state exists even though most researchers have limited themselves to studying metal wear, and it is understandable that highway engineers have neglected analytical aspects of wear since pavement materials are much more complex than metals.

Since wear is concerned with the surface of a material, the contact area between two bodies is important. Enlargement of even the most carefully polished surfaces shows hills (referred to as asperities) and valleys which are large compared to molecular dimensions. Small particles, such as abrasives, dispersed between bodies also are called asperities in this paper. A second solid in contact with the first is supported on the summits of the highest of these asperities so that the area of actual contact is very small. This actual or real area of contact, A_r, is almost independent of the size of the surfaces and is determined by the load, W.

For loads which exceed the yield point, the deformation is plastic and

$$A_r = \frac{W}{P_m} \tag{1}$$

where P_m is the material flow pressure. Measurements (1) of A_r show that even the lightest loads are sufficient to produce plastic flow.

The energy expended in overcoming friction between rubbing surfaces is dissipated in the form of heat. Since two surfaces touch only with small contact areas, extremely high temperatures may be reached at the contacting tips (2). Often the temperatures are only limited by the melting point of one of the surfaces.

Abrasive wear occurs when a rough hard surface or a soft surface, containing hard particles, slides on a softer surface and plows a series of grooves in it. The material is thought to be gouged out of the grooves to form loose wear particles. Such gouging involves local deformation. The resistance to deformation is commonly called hardness. To measure it a hard indenter may be pressed into the surface with a known load, and the size of the indentation is measured. The Vicker's indenter, a square pyramid of diamond, is frequently employed in metal hardness determinations. The impression is permanent since the overriding effect is the plastic flow of the metal around the indenter. The Vicker's hardness number, the mean pressure over the area of the indentation, is expressed as

$$H_v = \frac{\text{load}}{\text{projected area of indentation}}$$

It can be shown (3) that

$$H_v \sim P_m \tag{2}$$

a desirable relationship. Other testers or methods to measure hardness, such as those bearing the names of Brinell, Rockwell, or Mohs, employ scales which vary as some power of the flow pressure.

Avient, Goddard, and Wilman (4) abraded a number of metals using a wide size range (5 to 150 μ) of emery abrasive. If v is the volume loss for one particle per unit distance of travel, $v = (A_V - A_O)$, where $A_V = f(A_r)$ is the area of the verticle face of the wear track and A_O is the equivalent area of the piled-up ridge along the groove. Since the frictional force $F = P_m A_V$,

$$v = \frac{F}{P_m}\left(1 - \frac{A_O}{A_V}\right) \tag{3}$$

Assuming that A_O/A_V is constant for most metals, and using the fact that $F = fW$,

$$v = \frac{c_1 W}{P_m} \tag{4}$$

where the coefficient of friction f depends on the shape of the abrasive particle, but is independent of its size. Mulhearn and Samuels (5) tested silicon carbide abrasive on metals, and their results agreed with Eq. 4.

The abrasive wear in these tests was of the two-body type (abrasive fixed to an adhesive backing), but Rabinowicz et al (6) experimented using a three-body geometry, i. e., the abrasive is loose. The wear was an order of magnitude lower than with two-body abrasion since the grains were rolling about 90 percent of the time. However, the results agreed also with Eq. 4.

Spurr and Newcomb (7) slid various metals against emery paper. The wear of these metals was inversely proportional to the elastic modulus and did not correlate as well with hardness. Microscopic examination revealed that, when a surface is pressed against emery paper and moved a small distance, the abrasive plows through the surface, but no wear particles are formed until sufficient sliding has occurred for a new groove to run into an earlier produced one. When the first abrasive grain slides along the surface, it displaces metal ahead of it, but the metal behind it recovers elastically. The second grain removes the recovered material and the volume removed was related to elasticity:

$$v \sim \frac{W}{E} \tag{5}$$

Selwood (8) abraded metals as well as nonmetals against carborundum paper (60 μ). He found that extensible or elastic solids were abrasion resistant and that hardness was a minor factor. For instance, antimony is three times harder than cadmium yet it was abraded more rapidly.

Rabinowicz (9) proposed an interesting theory for particle size formation based upon energy concepts. If a particle breaks loose beneath an asperity the elastic energy stored in the particle while it was being formed must equal or exceed the energy of adhesion which binds it to its substrate. Let T = elastic energy and Y = energy required to create a new surface, then for a hemispherical fragment of diameter d,

$$T \sim \sigma_{yp}^2 \frac{d^3}{E} \tag{6}$$

$$Y \sim Sd^2 \tag{7}$$

where σ_{yp} is the yield stress and S is the surface energy per unit area. Thus,

$$T \geq Y \text{ or } d \geq \frac{c_2 ES}{\sigma_{yp}^2} \tag{8}$$

This parameter may provide a measure of wear resistance although there is no indication of average size or frequency of wear particle detachment. For instance, one of the first useful empirical rating of wear resistance, 1/v, (10) was the property σ_{yp}/E. This property does give some measure of a material's ability to store strain energy upon deformation.

Most of the above studies were concerned primarily with the wear of metals. Some nonmetallic materials, in particular minerals, are known to possess bulk properties which make them much more brittle than metals. King and Tabor (11) investigated the sliding contact regions of brittle solids and found that the high pressure developed around the deformed region were often sufficient to inhibit brittle fracture. Under these conditions the deformation is primarily plastic, although some cracking and surface fragmentation occurred.

The abrasion of graphite with emery paper (5 to 150μ) by Porgess and Wilman (12) was similar to that of Eq. 4; however, wear did not vary linearly with the friction coefficient f, or abrasive size. Microscopic inspection revealed cracking and fragmentation along the wear grooves. For emery particles larger than 50μ the wear was about four times as much as predicted from metal wear theory. Dobson and Wilman (13) continued the abrasion tests of nonmetals wearing sodium chloride against emery abrasive (0.5 to 150μ). The results agreed with those obtained with graphite in that the proportion of the groove volume removed as wear due to brittle fracture increased with abrasive size. One important new feature was reported though. In the lowest region of abrasive particle size, where a large number of particles share the load and indentations are shallow in the specimen, wear characteristics agree with the studies done on metals. Two types of wear then are distinguished and are thought to be operative in different ranges of the depth of indentation. At depths less than 0.5 μ, fractures and cracking are negligible, and wear is identical in magnitude to that of metals of the same hardness. In this instance the deformation is entirely plastic. For depths beyond 5μ, the wear increases strongly with increasing spread of fractures around the indenting abrasive particle. No attempts were made to relate fracture wear to strain energy properties.

Previously it was noted that frictional heating is often limited only by the melting point of the rubbed surface, particularly for materials with low thermal conductivity. In addition the mechanical strength of most materials at high temperature declines rapidly near the melting point.. Can melting temperature than be expected to give a measure of wear between material pairs?

The influence of melting point is very pronounced in work by Bowden (14) where metals were worn with a block of pure camphor (melting point 178 C). The observed wear is given in Table 1. The loss of weight primarily depended on melting point, not hardness. If the melting point of the rubbing material is lower than the rubbed material, the rubbing material will be relatively ineffective. For example calcite (melting point 1330 C) which showed little wear when rubbed with cuprous oxide (melting point 1230 C) was readily worn by zinc oxide (melting point 1800 C). Quartz (melting point 1700 C), which is considerably harder than zinc oxide, was worn by it.

Bowden and Scott (15) studied the wear of glass due to a diamond slider. Wear was negligible below a critical value of $VW^{1/2}$ where V is the velocity. Surface examination revealed melting, and $VW^{1/2}$ was related to the melting temperature of the particular glass. The polishing of glass with diamond dust gave the same type of surface deformation as the diamond slider.

TABLE 1

CAMPHOR BLOCK SLIDING ON METALS

Metal	Melting Point (°C)	Vicker's Hardness (kg/mm²)	Loss of Weight (gr/cm)
Lead	327	5	$<0.1 \times 10^{-7}$
Wood's alloy	69	25	3.2×10^{-7}
Gallium	30	6.6	165×10^{-7}

This abridged review of wear theory highlights the complexity of wear processes through simplified models and their empirical disagreement. One reason for these discrepancies is the degree of correlation that often exists between the macroscopic properties of hardness, melting temperature and modulus, which ultimately depend upon the atomic structure of the material. Thus, strong atomic bonding promotes high values for these variables. However, relations between wear and more fundamental properties have not evolved.

PROCEDURE OF THE INVESTIGATION

Experimental Variables

It is commonly accepted that the tire-pavement wear process is essentially of the three-body abrasion type. Abrasives, present on the road, are probably derived primarily from the pavement itself. Rolling tires provide enough relative motion to abrade the exposed aggregate. To learn more about this process a limited microscopic study was made of the abrasives found on the road and tires of vehicles. The collection was done with adhesive tape. Attention was given to size rather than type. The debris collected from the road surface was dominated by particles in the 5- to 40-μ range. Tires were found to be coated with an extremely fine powder, dominated by 1- to 10-μ particles. Roadstones, that had been exposed to traffic, are quites smooth in appearance. A number were collected, and the exposed surfaces were examined further. The observed scratch lines could be produced only by abrasives less than 10-μ diameter, presumably the debris clinging to the tires.

There are numerous physical properties and test variables that could play a part in the wear process. One would like to investigate as many of these properties as possible and yet keep the scope of the experiments in bounds. The following selections were made:

1. Ten minerals;
2. Three abrasives;
3. Three loads; and
4. Three speeds.

The minerals were selected for consistent and known physical properties. Mineral oxides were the primary choice. In this respect, the selection differed somewhat from actual paving practice, although quartz, slag, and limestone were included. The final choice was a compromise between variability of properties and availability of material. The abrasive selection was a similar compromise: oxides of aluminum, silicon, and magnesium. Size was specified as less than 10μ. A complete list and source of the minerals is given in Table 2.

A rolling tire can produce lateral movements of the order of ¼ in. in the contact zone (16). Using a value of 8 in. for the tire contact length and a vehicle speed of 60 ft/sec, one arrives at 3 ft/sec for these contact velocities. A selected speed range (1 to 6 ft/sec) therefore is considered realistic. Loads were chosen to give

TABLE 2

MINERAL AND ABRASIVE SOURCE

Substance	Name and Source
	(a) Mineral
MgO	Magnorite crude, Norton Co., Cippawa, Ontario
ZrO_2	Zirconia H (⅜ in. and finer), Norton Co , Cippawa, Ontario
Al_2O_3	Alundum (No. 4 mesh), Norton Co., Cippawa, Ontario
SiC	Crystolon (½ in lump), Norton Co., Cippawa, Ontario
Slag	Assorted chips for pavement, U.S Steel Corporation, Pittsburgh, Pa.
SiO_2(f)	Clear fused quartz rod (6-mm dia), Engelhard Industries, Inc., Newark, N.J.
$CaCO_3$(l)	Limestone chips (No. 4B), Metal Finish, Inc., Newark, N J.
$Al_6Si_2O_{13}$	Shamva mullite chips (¼ to ⅜ in.), H. K. Porter, Co., Shelton, Conn.
SiO_2(c)	Glass-like lump, Earth and Mineral Sciences Dept.
$CaCO_3$(c)	Crystal-like with cleavage planes, Earth and Mineral Sciences Dept.
	(b) Abrasive
MgO	Magnorite Type II, Norton Co , Cippawa, Ontario
SiO_2	Microsil silica sand (35 percent smaller than 7 μ), Standard Silica Co., Ottawa, Ill.
Al_2O_3	Alundum No. 38 (900 mesh), Norton Co., Cippawa, Ontario

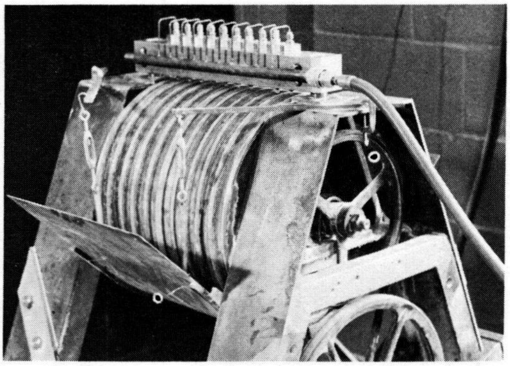


Figure 1. Test drum and assembly.

contact pressures above and below normal tire inflation pressure (10 to 50 psi). Higher pressures were avoided since they caused tearing and wear of the rubber carrier.

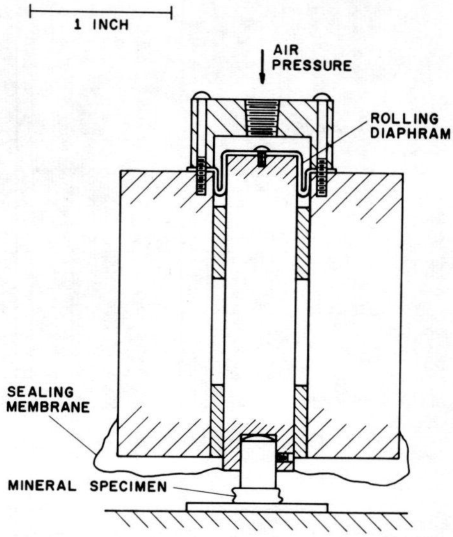


Figure 2. Loading plunger with specimen holder.

Apparatus

The apparatus is shown in Figure 1. A channel iron frame supported a 14-in. diameter steel drum to whose outside strips of rubber tape were applied. The drum could be rotated at any desired speed. Ten ¾-in. strips of rubber tape were applied to the drum to rub against the ten mineral specimens. A V-shaped bin for holding abrasive was placed around the lower half of the drum. Two springs urged the bin against the rubber strips so that the rotation of the drum produced a steady abrasive coating.

A block above the drum held ten radially movable plungers to which the specimens were fastened. The plungers were urged by air pressure toward the drum. A rolling diaphragm served as seals (Fig. 2). Mineral specimens consisting of cubes of about ¼-in. side length were bonded to the end of steel plugs. The plugs were held in the plungers by set-screws (Fig. 2). The moving parts were protected from the fine abrasive by a thin plastic membrane.

Wear was determined as weight loss; to this end the complete specimen-plug assembly was weighed repeatedly on an analytical basis. Weight loss was converted to volume loss using material density.

An exploratory program was carried out with SiO_2 abrasive to study the wear effects of changes in the apparent contact area. From friction and wear theory dependency is not expected. Minerals of $CaCO_3$, MgO, and Al_2O_3 were included. A typical example of the results is shown in Figure 3. Wear was independent of apparent contact area for a given load. Similar behavior was assumed to hold for all the remaining mineral-abrasive combinations.

The majority of abrasion tests reported by others showed wear to be proportional to distance of travel. This fact was also verified in the preliminary tests. Thus, the distance of travel (16, 200 drum revolutions) was the same for all tests, and an adjustment of running time was made at each speed.

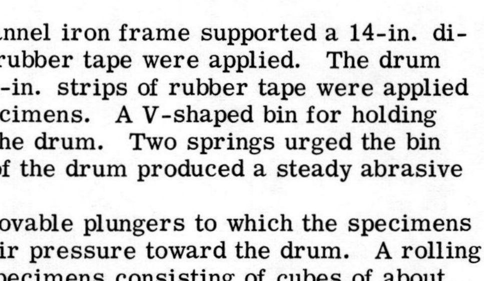
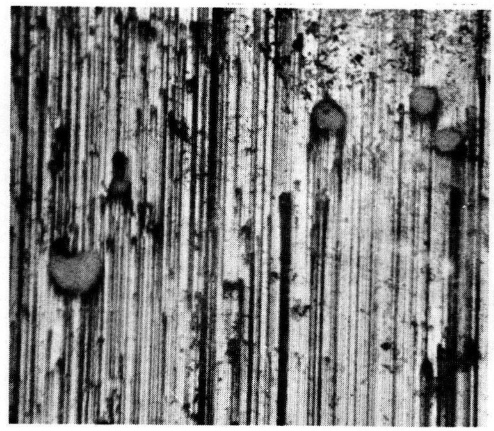
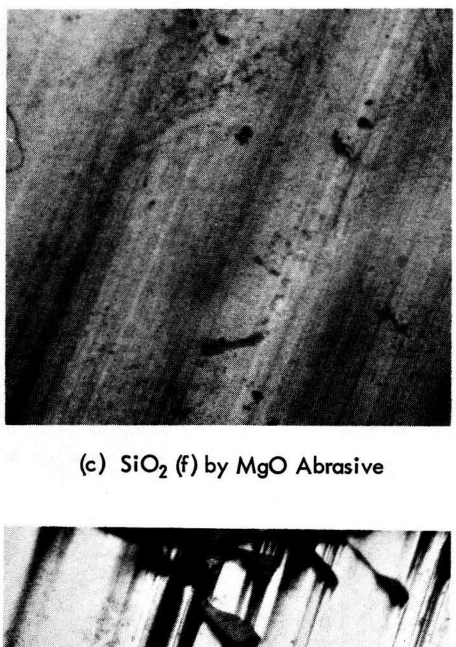


Figure 3. Wear of MgO as a function of apparent contact area.

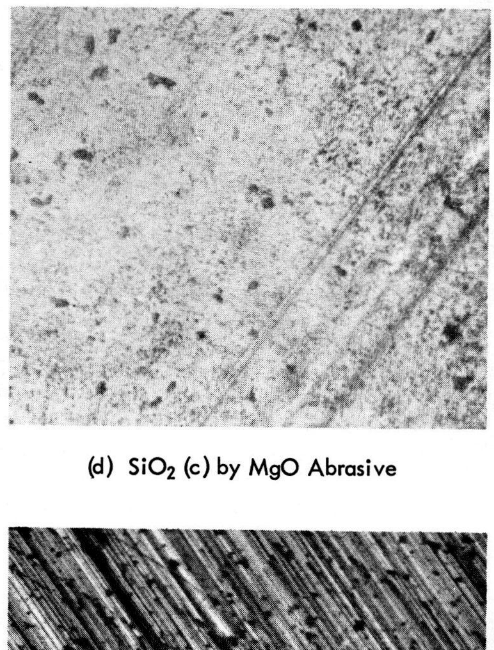
(a) $CaCO_3$ (c) by MgO Abrasive



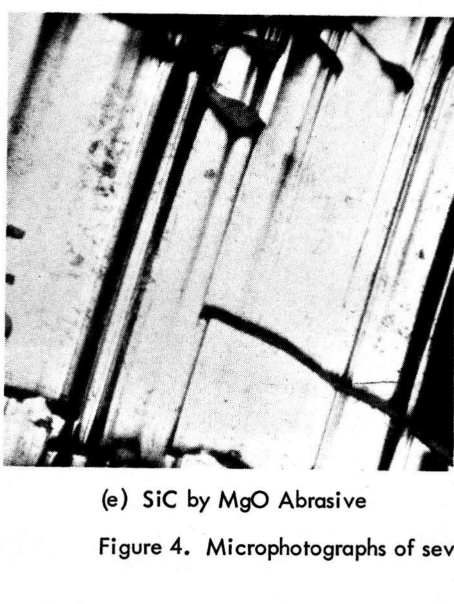
(b) Slag by MgO Abrasive



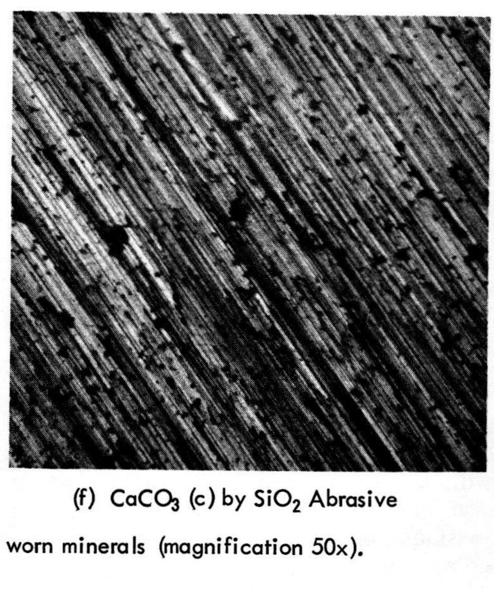
(c) SiO_2 (f) by MgO Abrasive



(d) SiO_2 (c) by MgO Abrasive

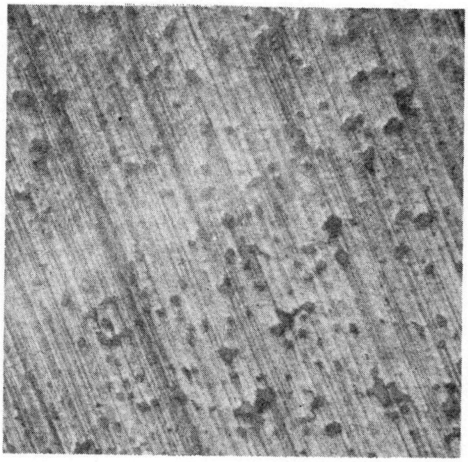


(e) SiC by MgO Abrasive



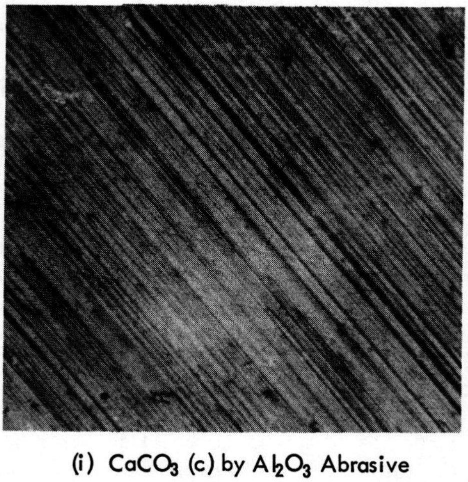
(f) $CaCO_3$ (c) by SiO_2 Abrasive

Figure 4. Microphotographs of several worn minerals (magnification 50x).

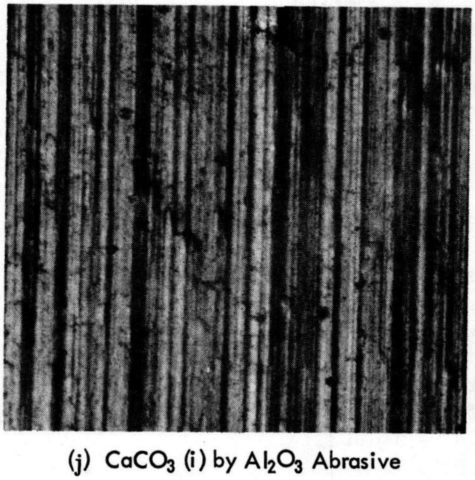


(g) Mullite by SiO_2 Abrasive

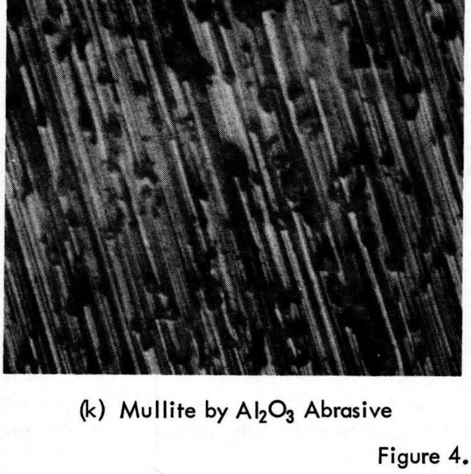
(h) Slag by SiO_2 Abrasive



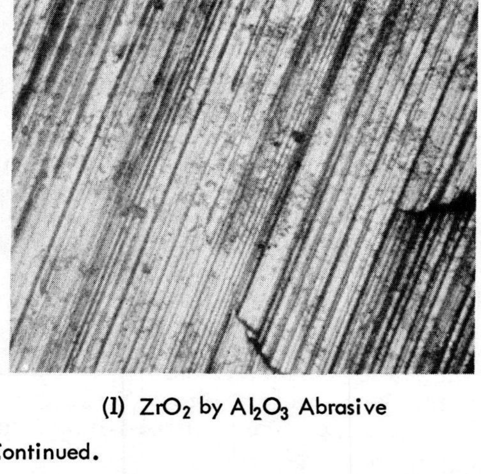
(i) $CaCO_3$ (c) by Al_2O_3 Abrasive



(j) $CaCO_3$ (i) by Al_2O_3 Abrasive



(k) Mullite by Al_2O_3 Abrasive



(l) ZrO_2 by Al_2O_3 Abrasive

Figure 4. Continued.

TABLE 3

LIST OF MINERAL PROPERTIES WITH AVERAGE WEAR

Mineral	Melt Temp. (°C)	Modulus (psi × 10^6)	Yield Stress (psi × 10^6)	Specific Gravity	Vicker's Hardness (kg/mm^2)	Abrasive Wear ($mm^3 \times 10^{-3}$)		
						Al_2O_3	SiO_2	MgO
$CaCO_3$(c)	825	–	–	2.70	460	42.2	30.7	13 5
$CaCO_3$(i)	825	–	–	2.70	400	38 0	40.9	12.5
Slag	1400	–	–	2.70	620	24.0	16.3	9.4
SiO_2(f)	1700	6	–	2.10	1100	11.0	11.0	2.6
Mullite	1810	21	0.9	2.95	1720	11.0	6.5	3.1
SiO_2(c)	1700	7	0.7	2.65	2000	7.7	6 5	1.7
MgO	2620	42	1 5	3.60	1240	3 4	1.3	0.2
ZrO_2	2650	21	2 0	5.70	1700	2.4	0.6	0 2
SiC	2200	50	5.0	3.00	4500+	0.7	0.4	0.07
Al_2O_3	2000	45	4.0	4.00	3300	0.3	0.7	0.03

RESULTS

It was seen that numerous material properties, notably melting temperature, hardness, modulus and strain energy, could provide a means to predict the degree of wear between mineral pairs. In particular the wear concepts throw doubt on the usually accepted parameter, scratch hardness. Table 3 lists these properties and gives the wear (average of all loads and speeds) for each abrasive. Hardness was measured directly by the author. The remaining values were found from an assortment of publications on material constants.

A correlation of wear and melting point was attempted initially. At first glance the postulated correlation seems to hold: low melting point materials tend to have high wear. The highest melting point materials (i. e, MgO and ZrO_2), however, do not exhibit the best abrasion resistance. In fact, closer inspection of the table reveals numerous exceptions. Al_2O_3 (2000 C) abrasive wore MgO (2600 C) and ZrO_2 (2650 C) much faster than it wore SiC (2200 C) or itself; MgO (2600 C) abrasive wore ZrO_2 (2650 C) much faster than it wore SiC (2200 C) or Al_2O_3 (2000 C). Also the wear rates of fused and natural quartz differ by almost 50 percent though both have the same thermal properties. Apparently melting point does not provide a unique measure of wear.

A relation between wear and elastic modulus does not appear to be satisfactory. Mullite is much more brittle than SiO_2(c) but the amount of wear is similar. The same is true for MgO and ZrO_2.

Comparing wear with hardness in Table 3, the following generalization can be made: for those minerals softer than the abrasive, the amount of wear is large; if the mineral are harder than the abrasive, the wear is an order of magnitude lower than the softer materials. Several worn samples were preserved after the experiments and microphotographs were taken of their surfaces. The surfaces are shown in Figure 4. Two phenomena can be seen: (a) scratching or grooving, and (b) pitting or scabbing. The scratches appear to be well formed; little, if any, fragmentation occurs at their edges. Scratches on minerals harder than the abrasive are considerably reduced from those scratches on the softer minerals, see Figure 4 c, d, e. Pitting is found on all mineral regardless of their hardness relative to that of the abrasive. The number and size of the pits differ from mineral to mineral and the pits seem little affected by the type of abrasive used.

Low Hardness Minerals

Abrasive wear theories for metals suggest that wear can be caused by plastic deformation of the contact and subsequent plowing-out of wear debris when movement commences. Wear was found to be directly proportional to load and inversely proportional to flow pressure. Porgess and Wilman showed that this relation holds for rock salt.

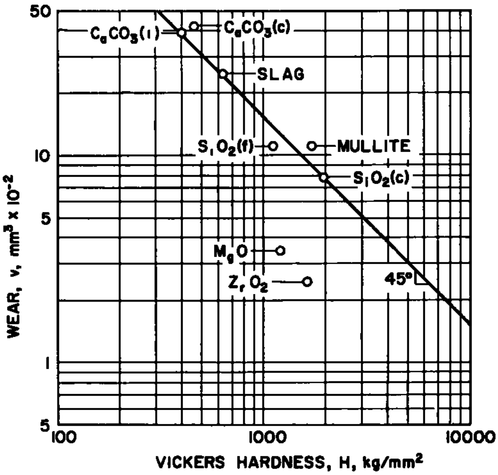


Figure 5. Wear inversely proportional to hardness for Al_2O_3 abrasive.

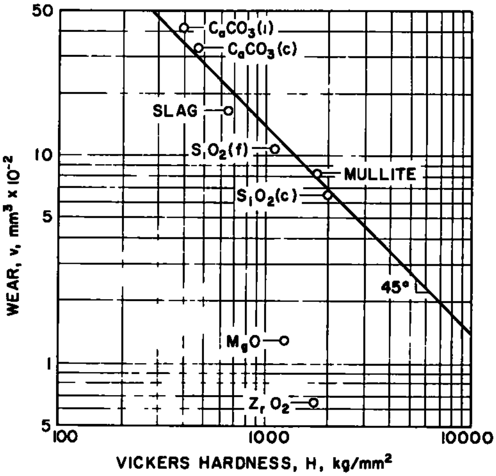


Figure 6. Wear inversely proportional to hardness for SiO_2 abrasive.

This agreement with metals may be expected since salt is an extremely soft mineral. No other experiments were found which directly verify these proportionality relationships for nonmetals.

The minerals used in this research are much harder than salt, still no fragmentation of the grooves is evident from the photographs. Thus one might expect the wear to be inversely proportional to the flow pressure or Vicker's hardness. A log-log plot of wear against hardness was made for each abrasive, Figures 5 through 7. In each case the wear was generally found to be inversely proportional to the first power of hardness (in Fig. 7 SiO_2(f) is nearly as hard as the abrasive). But exceptions occurred for the SiO_2 and Al_2O_3 abrasives. Both MgO and ZrO_2 gave less wear than the hardness-wear relationship for the other materials would predict.

Figure 8 shows the effect of load on wear. Wear increases with the load, provided the mineral is softer than the abrasive. From abrasive wear theory, wear should be

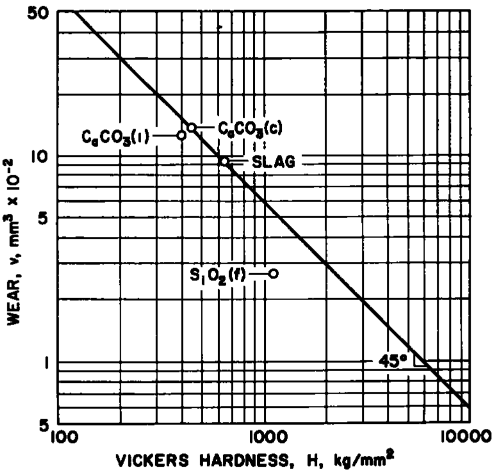


Figure 7. Wear inversely proportional to hardness for MgO abrasive.

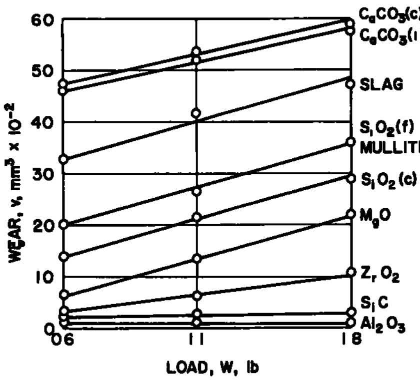


Figure 8. Wear as a function of load for Al_2O_3 abrasive.

proportional to load. The graph indicates a load independent intercept W_0 that is not accounted for in the theory. Its value increases as the mineral hardness decreases. The constant term W_0 apparently is related to the pitting. This matter, as well as the behavior of MgO and ZrO_2, is discussed further in the following section.

Melting point does give an indication of a material's high temperature strength. The room temperature hardness might be lowered to some degree by the heat generated at the rubbing contacts. Heat generation depends on the relative speed of the rubbing surfaces. Higher speeds should, therefore, lower the instantaneous hardness and increase wear. Figure 9 is an example of the wear for each abrasive as function of speed. The results show a definite increase in wear as the sliding speed increases. The slope for Al_2O_3 abrasive increases in proportion to the decrease in mineral melting point (Table 3).

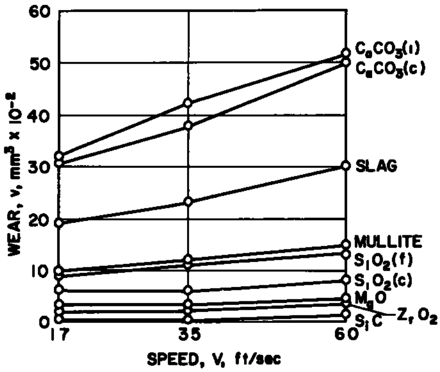


Figure 9. Wear as a function of speed for Al_2O_3 abrasive.

High Hardness Minerals

The wear of the minerals harder than the abrasive was relatively low. It was also independent of load. Microphotographs revealed that grooving was minimal although some pitting remained. The appearance of the surfaces suggested applicability of Rabinowicz's theory of a balance between strain energy and surface energy. The diameter of a detached particle is given by

$$d \geq \frac{EH_V^{1/3}}{\sigma^2_{yp}} \tag{9}$$

where the surface energy S is approximated by the ⅓ power of the hardness, valid for many materials according to Rabinowicz. The theory does not give the average size of the particles nor does it indicate the frequency of detachment. Values of the ratio in Eq. 9 computed for several materials are mullite, 3; MgO, 2; SiO_2, 1.8; and Al_2O_3, ZrO_2, and SiC, 0.5. Visual inspection of several microphotographs, Fig. 4 g, k, d, (1), shows that the average size and frequency of detachment are in the same order as these ratios. Thus it may be possible to relate wear to the same ratios.

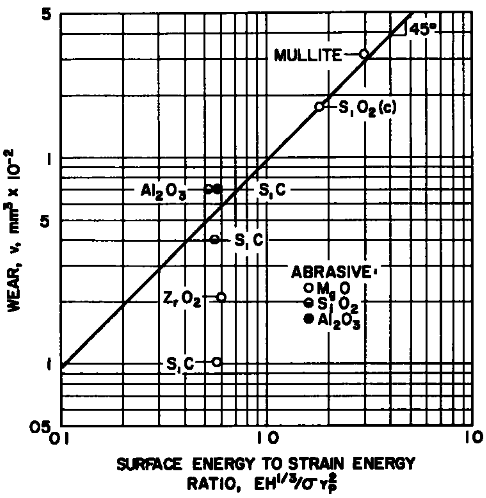


Figure 10. Relation between slow wearing minerals and energy parameter.

Wear for minerals harder than the abrasive is plotted against $EH_V^{1/3}/\sigma^2_{yp}$ (Fig. 10). The plotted points were obtained with the three different abrasives. The evidence is that the wear may be directly proportional to this ratio although more data are desirable. If the mineral is considerably harder than the abrasive, e. g., MgO, the wear is less than the value this relation would be expected to provide. Referring to Figure 4 e little pitting occurs with the MgO abrasive. Perhaps relative hardness has some influence on the frequency of detachment. Rabinowicz has verified that minimum particle size does obey Eq. 8 but no attempts have been made by anyone to evaluate volume of particle removal in terms of the parameter.

The microphotographs show substantial pitting for the minerals softer than the abrasive. Since the pitting type of wear appears independent of load, it seems plausible to relate it to the constant W_O found for these minerals. According to Rabinowicz, the substitutions $E \sim 1/H_V$, $S \sim H_V^{1/3}$, and $\sigma_{yp} \sim H_V$ are valid for many materials. Thus, the size of the detached particle can be described by

$$\frac{ES}{\sigma_{yp}^2} \sim H_V^{-3/4} \tag{10}$$

Hardness can therefore be taken as a measure of the particle detachment size when the terms on the left of Eq. 10 are not known. Indeed, the microphotographs for the softest minerals, $CaCO_3$ and slag, show a higher number of detachments and a larger increase in their size compared to harder minerals. In Figure 9, W_O does increase in proportion to the softness of the minerals.

Eq. 10 was given because the properties on the left side are not always known. That the pitting phenomenon should be related to hardness is not surprising since good correlation of hardness with wear was achieved for the softer minerals. Eq. 9 was used for the studies of minerals harder than the abrasive because hardness was not a good indication of the particle detachment diameter as seen from the ratios above. Values for ZrO_2 and MgO were comparable to SiC and SiO_2(c), even though the latter are much harder. If there is less pitting for ZrO_2 and MgO than their hardness indicates, it would explain why the wear of these minerals did not correlate better with hardness when worn by a harder abrasive. The available microphotograph for ZrO_2 does show much less pitting than materials of comparable hardness. Figure 9 also shows that the term W_O for ZrO_2 and MgO is small is view of their hardness.

CONCLUSIONS

Wear concepts provided several parameters that could be used to predict the wear of homogeneous minerals or roadstones under simulated road conditions. The results of relating wear to mineral melting temperature or modulus were negative.

Microscopic photographs of worn surfaces revealed two phenomena: scratching and pitting. No cracking from brittle fracture was evident along the scratches. Rapidly wearing minerals suffered both types of damage while slow wearing minerals displayed no scratching, only a small amount of pitting. The criterion which established whether or not a mineral would wear rapidly was its hardness relative to that of the abrasive. Rapid wear occurred when the abrasive was harder than the mineral. For each of the three fine abrasives the rapid type of wear correlated well with mineral hardness with the exception of two minerals which appeared to have little pitting.

The pitting that is evident in the microphotographs strongly suggests some energy mechanism involved with the material removal. However, no adequate model is known which will relate, quantitatively, the volume of wear to the energy properties of minerals. Qualitative relations were developed to explain the behavior of the above two minerals and the slow wear of minerals harder than the abrasive.

The significant aspect of this wear study is that hard minerals were worn with abrasives of comparative hardness; this combination has been neglected in wear literature. The evidence of scratching by the abrasive and the relation of rapid wear to mineral hardness is also a characteristic of the work reported for metals, but the phenomenon of pitting of the minerals which accompanies the scratching represents a new observation.

ACKNOWLEDGMENTS

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The opinions expressed in this paper are those of the author and not those of the supporting agencies.

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Pre-Evaluation of Pavement Materials for Skid Resistance—A Review of U. S. Techniques

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This paper is essentially a review of existing methods, used in the laboratory, for studying pavement materials as related to their skid-resistant qualities. It consists of descriptions of test equipment and testing techniques along with examples of data.

It reports that the laboratory tests have generally developed around two evaluation methods. One is to study compacted pavement mixtures; the other is to test aggregate particles or stone chunks. Both types of methods seem to be useful in evaluating pavement materials prior to their use.

None of the methods reported, however, have widespread use, nor are they entirely alike. Probably the Portland Cement Association and the Tennessee Highway Research Program equipment have the greatest similarity, but test procedures are different.

There are several new methods that are being constructed that include design ideas from older methods. Along with the mechanized laboratory test methods, information on supplemental pre-evaluation tools is reported, including procedures for studying the influence on skid resistance of sand-size siliceous particles, mineral content of aggregates, and pavement permeability.

It is concluded that the pre-evaluation of materials is a practical approach to obtaining better skid-resistant pavements, but additional work is needed to relate laboratory results to field performance.

•THE ability to pre-evaluate skid resistance of aggregates and their combinations prior to inclusion in a roadway surface is desirable. Pavement surface materials should be pre-evaluated for the safety of the driving public, as well as for economic reasons. If unsatisfactory materials can be eliminated, savings in accident costs, maintenance and reconstruction will result.

Presently available laboratory methods of pretesting pavement materials have generally developed around two techniques. One considers the complete mixture; the other considers mixture components. In addition to the laboratory measurement of pretesting pavement mixtures, other supplementary techniques have been used to provide a better understanding of the influence of aggregate particles and mixtures on slipperiness. These techniques include the determination of insoluable residue of aggregate particles, influence of surface weathering, and air and water permeability of pavement mixtures.

A review of the literature in this country reveals that there are several agencies that have made significant contributions to laboratory evaluation of pavement materials. They include Purdue University, Kentucky Department of Highways, the National Crushed Stone Association (NCSA), the Portland Cement Association (PCA), Pennsylvania State University and the Tennessee Highway Research Program. Several agencies including the states of California, Georgia, Maryland, Pennsylvania, and Virginia are extending or beginning preliminary work in establishing laboratory programs.

The presently known methods may be grouped on the basis of relative sample size. For example, the Georgia, Purdue and Kentucky apparatus use samples between 4- and 6-in. diameter, whereas the PCA and Tennessee apparatus use samples of sufficient size for testing with a standard size automobile tire. The NCSA, Maryland and Penn State methods utilize a circular track. In general, however, all methods require a conditioning of the surface, either before or during testing. This conditioning, sometimes referred to as polishing wear, is brought about by a rubber annulus in the case of Georgia, Purdue and Kentucky equipment and automobile tires in the case of the PCA, Maryland, Penn State and Tennessee equipment; a somewhat smaller tire is used by the NCSA. These laboratory techniques are sometimes referred to as "wear machines."

The following factors are reported by these agencies as relating to the materials problem:

For Aggregates	For Mixtures
Angularity	Surface texture
Gradation (size)	Horizontal drainage
Texture (micro and macro)	Vertical permeability
Mineral content	
Porosity	
Susceptibility to polishing	

PURDUE UNIVERSITY METHOD

The Purdue University method as described by Shupe and Goetz (1) and Stephens and Goetz (2) is adaptable to specimens molded in the laboratory or to ones removed from the highway surface. Specimen size is about 6-in. diameter.

The apparatus consists of a vertically mounted mandrel with a head sufficiently large to hold a 6-in. specimen (Fig. 1). A rubber test shoe is mounted on a ball-bearing shaft that is in line with the mandrel containing the specimen. This shaft is restrained from turning by a cantilever bar on which are located SR-4 strain gages. The restraint to rotation is recorded as the frictional resistance. In the test, the specimen is rotated at approximately 30 mph by an electrical motor and the rubber test shoe is forced against the specimen surface through a mechanical arrangement for applying a constant normal force of 28 psi. The applied force corresponds to the normal pressure one would expect to exist between passenger car tires and the highway pavement. The test shoe is held against the spinning specimen for 2 sec and then removed. After a pause of approximately 2 sec the test is repeated and the resistance is reported as a relative value. Water is supplied to the specimen surface during test.

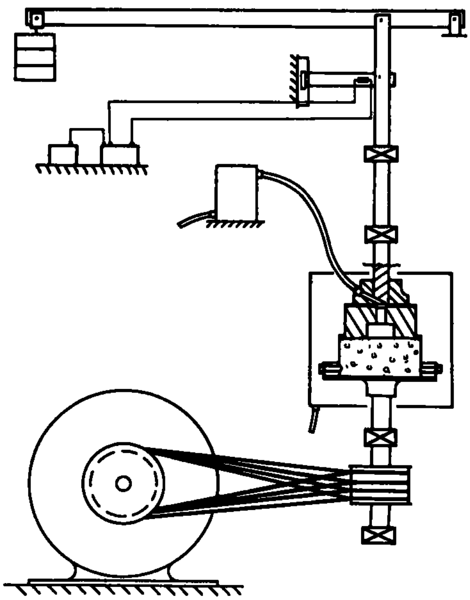

Figure 1. Purdue laboratory machine.

An attempt was made to correlate the laboratory results with field tests by using the stopping-distance automobile. In this correlation effort, the authors concluded that surfaces of medium texture had good

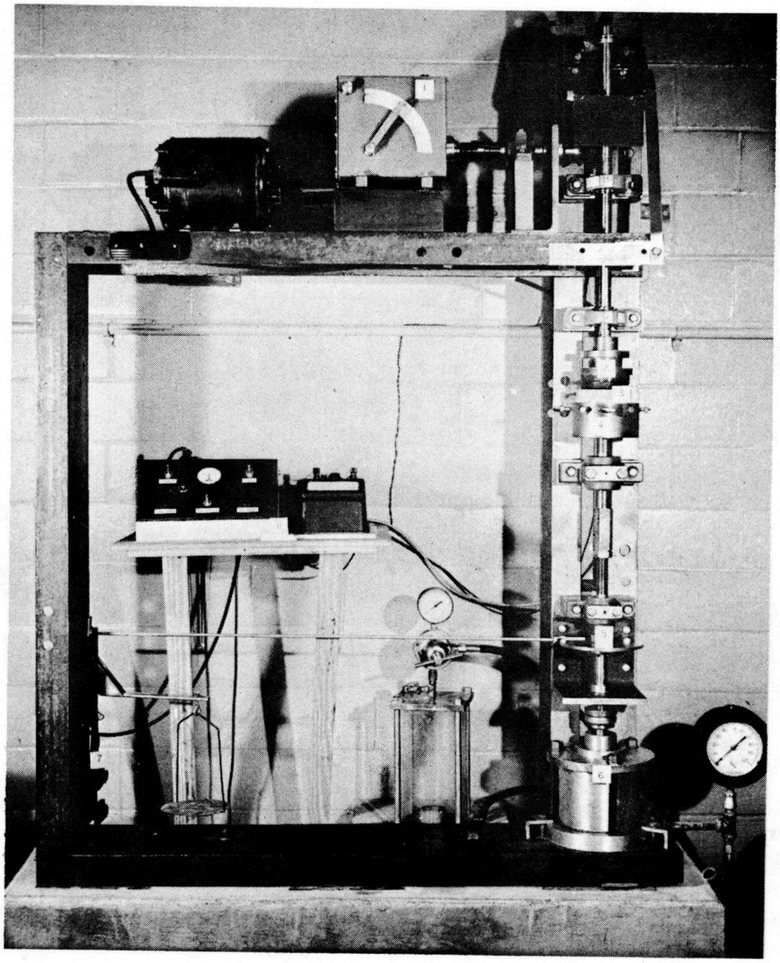

Figure 2. Kentucky friction measuring machine.

agreement between laboratory and field tests. In the open surfaces, the laboratory method indicated poorer anti-skid characteristics than did the stopping-distance method. For an extremely dense surface, the laboratory method showed higher values than obtained by the field measurements.

Typical results with this equipment demonstrate the capability of the method to discern the effect of replacing the fine aggregate fraction with the coarse material in asphalt or concrete mixtures. Results reported by Stephens and Goetz (2) also show the utility of the equipment for studying the relative resistance value of rock cores, aggregate blending, and the effect of aggregate texture. Supplementary research reported by Shupe and Lounsbury (3) relates to the use of the equipment for evaluating polishing characteristics of mineral aggregates. The aggregates were ranked relative to their skidding resistance by the test method.

KENTUCKY DEPARTMENT OF HIGHWAYS

The laboratory research reported by Stutzenberger and Havens (4) considered the testing of stone specimens that were controlled polished. It consists of measuring the resistance to turning of a rubber annulus when placed in contact with a polished stone surface. The device is shown in Figure 2. In operation, an electric motor furnishes the driving power, through a hydraulic torque converter, such that the rubber annulus

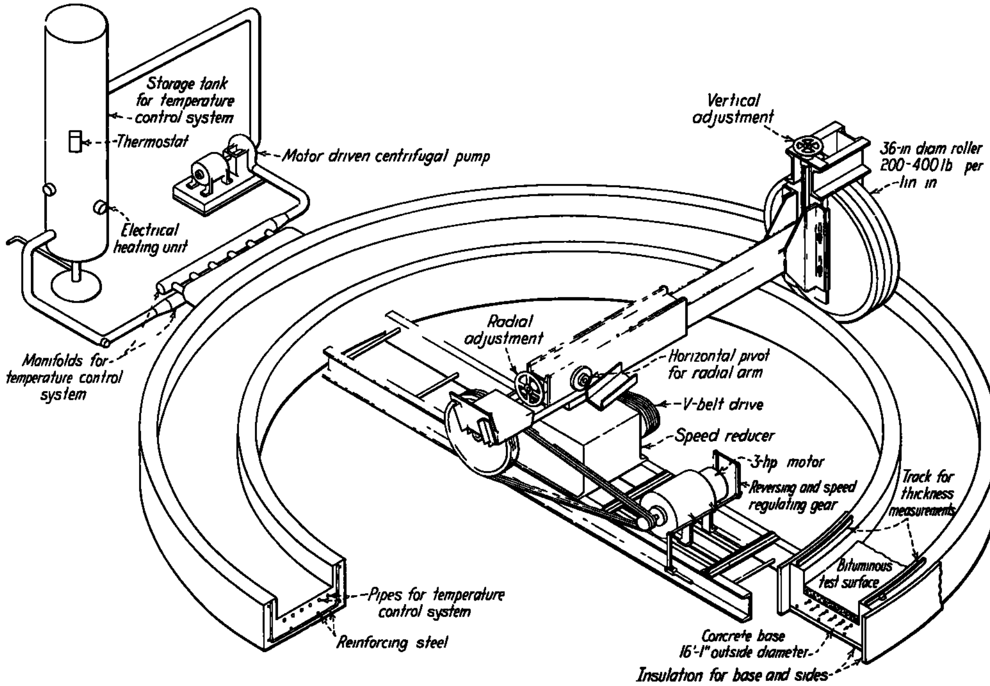

Figure 3. NCSA circular track.

may be rotated at speeds up to 300 rpm when in contact with the stone surface. The specimen supporting device is designed for use with specimens varying in length from 1 to 5 in. and a diameter of approximately 4 in. The loading mechanism consists of a pneumatic cylinder that can provide a normal load on the specimen from 0 to 32 psi. A strain gage bar is attached to the unit that holds the specimen so that when the shaft rotates, the resistance to rotation is transferred to the strain gage bar and is automatically recorded.

The original work using this equipment involved the testing of specimens cored from large chunks of stone obtained from different quarries. These cored specimens were polished by grinding their sawed faces on a wheel that had been faced with an aluminum oxide paper. After initial grinding, the specimens were then ground on a glass plate in a slurry of coarse carborundum and then with the fine grit carborundum. The polishing was continued until the surface was uniform and smooth, after which further grinding was done on another glass plate with a different slurry. The final polishing was accomplished by a buffing wheel. This final stage polishing was continued until three consecutive readings on a reflectometer remained unchanged. After polishing, the stone specimens are tested in either the wet or dry condition. The authors report a method for calculating the coefficient of friction from the test procedure.

GEORGIA HIGHWAY DEPARTMENT

The apparatus being used by Georgia (5) is similar to the Purdue machine, but uses a 10-in. diameter specimen. The purpose for the larger specimen is to permit the use of a British portable tester for the measurement of wear. Also, the Georgia method utilizes a water-abrasive mixture to accelerate the wear process. The apparatus has been in experimental use since 1967.

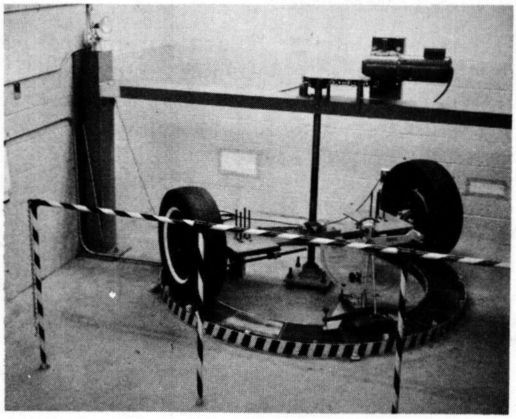

Figure 4. Maryland circular track.

NATIONAL CRUSHED STONE ASSOCIATION

Gray and Goldbeck reported (6) on the NCSA method of pre-evaluation of pavement mixtures. This method also involves polishing the specimen before testing for frictional resistance. A circular track is used for containing the pavement surfaces while they are being polished. The surfaces may be fabricated in the laboratory or obtained directly from the field.

The track is 14 feet in diameter and will accommodate up to 20 different test sections (Fig. 3). The surface preparation consists of rotating a pneumatic-tired wheel many times around the tract, first with water and fine sand on the surface and finally with the surface clean and dry so that only the rubber tire exerts the polishing action. It is believed by the researchers that the circular track technique produces surfaces that closely resemble those formed by normal vehicular traffic.

After the specimens have been polished, measurements of slipperiness are made with the NCSA bicycle wheel. This wheel is supported in a frame and is placed directly on the pavement section to be tested. The tire is ground off exposing the tire fabric over half of its circumference; the other half of the tire retains its full thickness of tread. Slipperiness is measured by rotating the tire to bring counterbalance weights attached to the rim to the uppermost position and the height of the wheel is adjusted so that the wheel turns freely except when the thick portion of the tire is down on the surface.

The wheel is released allowing the weights to rotate it so that the thick portion of the tire strikes the pavement surface, thereby raising the wheel slightly in the slotted supports which hold the axle. The wheel is then supported only by the surface, and it continues to turn until brought to rest by the friction between the tire and the road. The more slippery the pavement the greater is the angle of turn required to bring the wheel to rest. An average of eight readings is taken in order to indicate the pavement slipperiness.

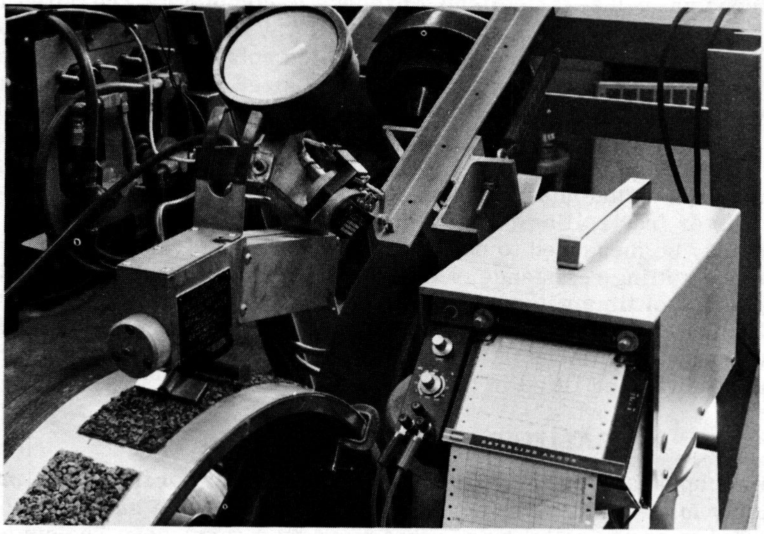

Figure 5. Penn State rotary wear machine.

MARYLAND STATE ROADS COMMISSION

The Maryland State Roads Commission (7) is moving into the laboratory for pre-evaluating pavement mixtures, and has developed an aggregate polishing device that has the features of a circular track (Fig. 4). Concrete as well as bituminous mixes may be tested. Test specimens are trapezoidal with approximate inside dimensions of 8 in. by 14 in. and 11 in. long. After the specimen surfaces have been polished, a British portable tester is used to evaluate their skid resistance. The circular track polishing device consists essentially of two wheels mounted 6 ft apart on a common axle. The wheels are rotated by a 3-hp gearhead motor at approximately 22½ rpm. The wheels can be adjusted to toe in, toe out, or rotate in a plane perpendicular to axle as well as track each other or run at slightly different radii. Provisions have been made to add surcharge weights to the wheels to attain a range of desired loads. The test tires are automobile size and have a slick tread.

In normal operation, four replicas of the same mix are tested. Two specimens are adjacent to each other and the other two are placed on the opposite side of the track. Generally the wearing process is stopped at selected intervals and each specimen tested with the British portable tester. Although only preliminary work has been done, the researchers have noted that with a typically poor aggregate the BPN number is 58 after about 1000 revolutions and this decreases to a BPN of about 49 after 40,000 revolutions.

The research is continuing with the initial intent of standardizing the process and establishing standard mixes against which others will be judged.

PENNSYLVANIA STATE UNIVERSITY

The Pennsylvania State University (8) also has three polishing devices, one of which is a circular track. One device has been referred to as a "reciprocating pavement polisher" which is used to polish individual aggregate particles that are mounted on a 12-in. square metal plate. Results with this device led the researcher to develop equipment to polish aggregates with a rotating tire. This device is referred to as a rotary wear machine (Fig. 5) and is believed to be more representative of the polishing process that occurs under traffic on the roadway. The machine operates by running an automotive tire against pavement samples that are mounted on the outside of a rotating drum. The test tire wheel is run against the drum at the chosen speed, load, and inflation pressure. Polishing agents are frequently introduced to accelerate the polishing wear. During test, the coefficient of friction is measured at intervals and its decrease is used as a measure of the progress of test. When the coefficient approaches a constant value, the polishing process is complete. The "drag tester" utilizing a BPN-type test shoe is mounted in such a way to permit measurement of the coefficient as the drum containing the test samples rotates. Test samples are 1 by 14 in. and consist of selected aggregates that are sized and glued to aluminum panels for attachment to the drum. The drum speed can be controlled between 30 and 50 mph. The other test parameters such as tire inflation pressure may be varied to suit test conditions. In addition to these two polishing devices, Penn State has a circular track (Fig. 6) which uses an automobile wheel. It can be operated under load at different amounts of slip or free rolling. Speeds are controlled from 5 to 24 mph. The torque on the wheel may be measured to obtain the friction force transmitted from tire to pavement or the rolling resistance of the wheel. It is designed to run continuously over a long period of time without attendance. The apparatus is used for wear test on tires or polishing of pavement surfaces and can be operated under controlled environmental conditions. Specimens of pavements may be made directly in the machine or cut from the roadway and fitted in the machine.

CALIFORNIA DIVISION OF HIGHWAYS

California's apparatus (9) may be used in the laboratory or field. It is presently used almost entirely in the field. The device (Fig. 7) consists of a small trailer-type tire that is mounted on two parallel guides that move on a carriage. The guides are rigidly

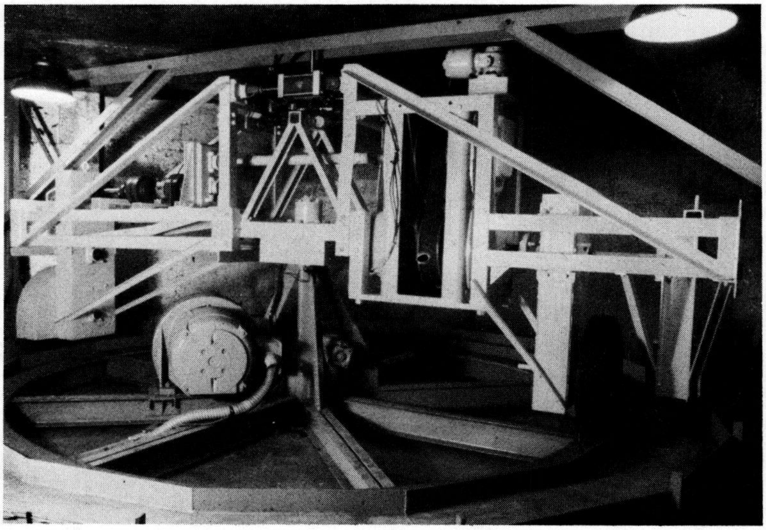

Figure 6. Penn State circular track.

connected into the frame of the assembly and are firmly fastened to a restraining anchor. In conducting the test the tire is brought to the desired test speed and is then dropped instantaneously to the test surface. Usually the test speed is 50 mph. In the pretest condition the tire is raised and adjusted to about ¼ in. above the test surface. The coefficient of friction is determined by reading the calibrated gage attached to the guide rods.

Although this method has been used to rate pavement surfaces in the laboratory, it is now almost entirely used in the field for locating and monitoring potential slick areas. In field operation it is attached to a bumper hitch of an automobile for proper positioning on the pavement.

TENNESSEE HIGHWAY RESEARCH PROGRAM

Whitehurst and Goodwin (10) reported on a device for determining the relative potential slipperiness of pavement mixtures prior to their use. It consists of an automobile wheel driven by an electric motor at a desired constant speed. With the test wheel spinning against a specimen surface, the power required to drive the wheel is considered to be a measure of the specimen surface resistance. As the surface becomes more slippery, less power is needed. Figure 8 is a general view of the test apparatus. It is used for testing specimens 6 by 8 in. up to 36 by 36 in. and 6 in. thick. Usually the specimens are about 24 by 24 in. and six tests are conducted on each specimen surface. Specimens may be fabricated in the laboratory or acquired directly from cut sections of the roadway. In addition to bituminous concrete, portland cement concrete, blocks of stone, and thin surface treatments may also be evaluated.

When in operation, the drive motor maintains a constant wheel speed of 10 mph under a normal pressure of 270 lb. The equipment is designed to operate unattended for conducting tests continuously for a long period. As a rule, however, four 1-hr tests constitute a "run" on each specimen surface. Generally speaking, no effort is made to provide accelerated wear of the pavement surface, although this can be done if so desired.

Test results are graphically presented with the ordinate of the graph representing power consumption of the motor with the wheel spinning against the specimen surface; the abscissa for the data graph is as test time. The chart paper of the recording wattmeter can serve directly as the graphical display.

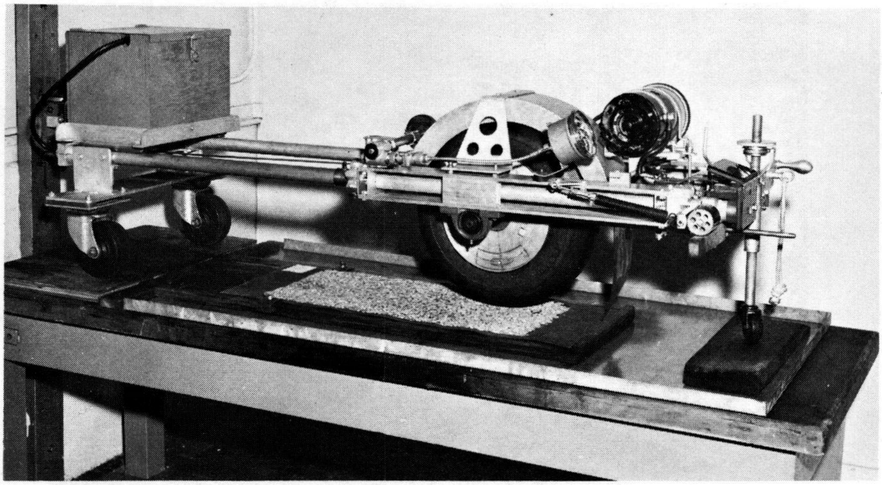

Figure 7. California slipperiness measuring device.

PORTLAND CEMENT ASSOCIATION

The laboratory studies of the Portland Cement Association are reported by Balmer and Colley (11). Their initial work was directed to the objective of determining the influence of aggregate properties on the skid resistance of concrete pavements. The test machine, patterned after the Tennessee machine, is shown in Figure 9. Essentially, it consists of a 25-hp electric motor mounted above an automobile differential assembly. The differential has been modified to provide a direct drive with the motor connected to the drive shaft through a series of belts and pulleys. The motor provides the power to rotate the wheel and tire against the test specimens. The tire is rotated at a constant speed of 20 mph and loaded with a normal force of 600 lb. The normal force is produced by deadweights mounted on a lever that acts against the supports for the test specimens. Water is conducted to the top of the test tire to aid cooling when the test is run continously. The ASTM Standard Tire for Pavement Tests (E249-64T) is usually used. During tests, an electric vibrator stimulates the flow of fine sand that

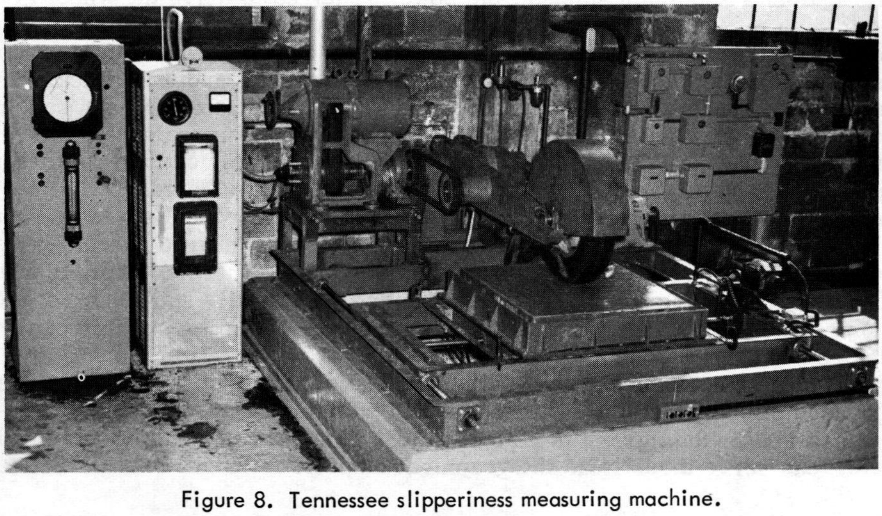

Figure 8. Tennessee slipperiness measuring machine.

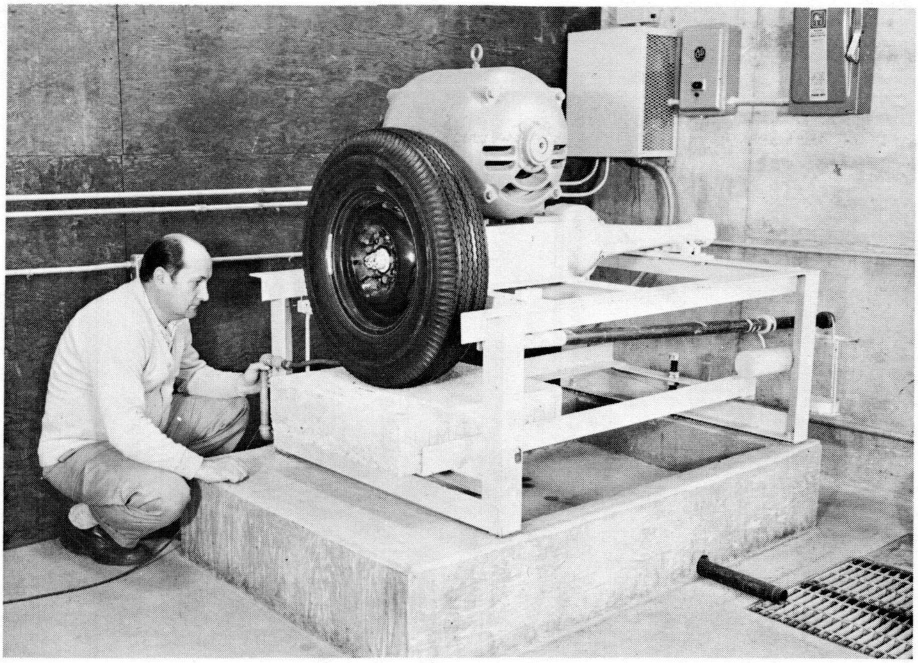

Figure 9. PCA slipperiness measuring machine.

is blown into the water stream to accelerate specimen wear. Test specimens in the order of 24 by 24 by 6 in. may be accommodated in the machine. They may be tested under either wet or dry conditions, but generally they are tested wet.

The testing procedure consists of placing the pavement specimen in the machine, turning the water on to keep the specimen surface continously wet, and energizing the electric drive motor to bring the test wheel to a speed of approximately 20 mph. After the machine is operated without load and the recording wattmeter is adjusted to zero, the specimen is lifted against the rotating tire and the normal test load of 600 lb is applied. During the first phase of the test, the tire rotates continously on the specimen for 75 min. In the second phase, a fine Ottawa sand is fed from the vibrator and blown onto the specimen so that it passes under the rotating tire that is in contact with the specimen. After two hours of sand abrasion, the third phase of the test is continued without sand for an additional 75 min. It is during this phase that the frictional resistance of the worn pavement is assessed.

Data from the test are in the form of a graph that relates a "wear index" with the time period of test. The wear index is considered to be a comparative measure of the skid resistance of the material, and is taken as the electrical power, in kilowatts, that is required to rotate the tire against the pavement specimen. Results are reported at the end of 270 min of testing. Such factors as concrete finish, exposed coarse aggregate, aggregate size, use of abrasives on concrete to develop frictional resistance, and the effect of different surface treatments in the restoration of skid resistance may be studied with this equipment.

Data developed in laboratory have been compared with field data and the authors observed that the wear index increased as field performance improved. They concluded "that the laboratory tests can be used to prejudge performance prior to using an aggregate in the field."

SUPPLEMENTAL PRE-EVALUATION TOOLS

In addition to the mechanized laboratory test methods, there are at least three other laboratory techniques that have the reported potential for providing supplemental in-

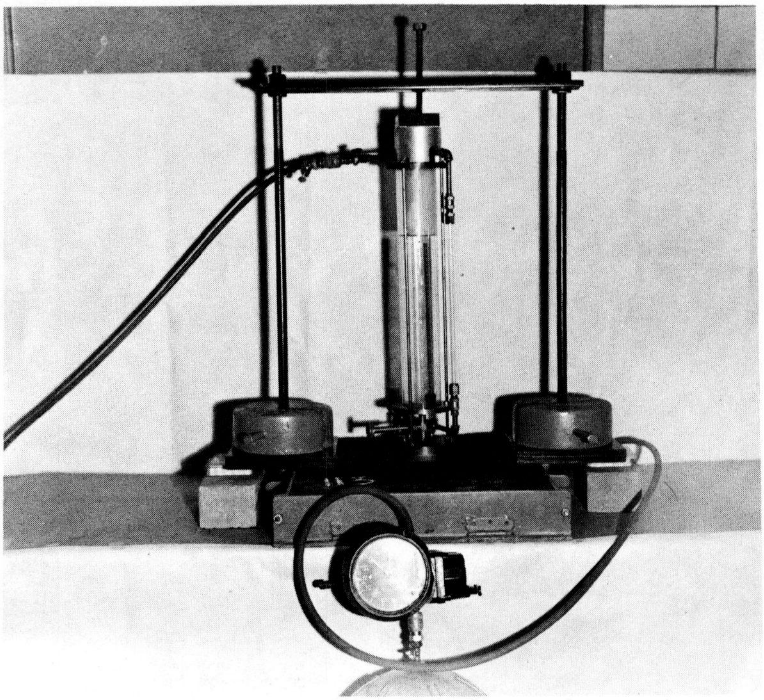

Figure 10. Tennessee water permeability device.

formation; these include the influence of sand-size siliceous particles based on the insoluble residue test, the influence of mineral content and nature of the total aggregate, and the influence of pavement permeability.

The work by Shupe and Lounsbury (3) along with that of Stutzenberger and Havens (4) related to studying the relationship between aggregate mineral content and susceptibility to polishing. Shupe and Lounsbury revealed that grain size and percent of calcium carbonate are useful factors in assessing the susceptibility of an aggregate to polishing under traffic wear. The research by Stutzenberger and Havens on specimens cut from blocks of sandstone and limestone points to the differences in coefficient of friction among these materials as influenced by coarse wear, differential hardness, cementitious material, and grain size. For example, the fine-grain dense stone polished more readily than the coarse-grain stone when subjected to Kentucky's testing techniques.

Gray (12), and later Gray and Renninger (13), reported that sand-size siliceous particles in an aggregate have an effect on skid resistance. The work reported by Balmer and Colley (11) and Goodwin (14) also supports this thesis. The procedures for determining the sand-size siliceous particles consist essentially of obtaining approximately 10,000 gm of the aggregate and soaking it in a dilute solution of hydrochloric acid until the acid has dissolved the carbonate minerals and there remains only a residue, which is filtered, washed, dried, and weighed. The residue is usually silt, clay, and siliceous material. The silt and clay, determined by washing the filtered solution over a 200-mesh sieve, are considered detrimental to skid resistance and are subtracted from the total residue to obtain the siliceous particle content. Balmer and Colley concluded that "the general trend of the data shows an increase in the wear index as the siliceous particle content increases."

The influence of pavement permeability, as affected by permeation of the water into the roadway surface and channelization over the roadway surface, is reported by Goodwin (14) as another measurable factor that can be used to aid in the pre-evaluation of pavement mixtures. The water permeability apparatus is shown in Figure 10. Typical data reported show that as the pavement water permeability increases so does the resistance to the turning of the laboratory test tire.

Other researchers, for example, Hutchison at the University of Kentucky and the late Pete Kummer at Pennsylvania State University have studied permeability as well as surface texture. The research at the University of Kentucky (15) is being used to determine the dynamic permeability of pavement mixtures. The research is in the developmental stage but initial work has involved laboratory studies for determining the amount of water that can be drained vertically into a given pavement beneath a vehicle tire at various speeds of travel. The device essentially consists of forcing the water into the pavement by a shotgun blast and measuring the water flow by volumetric means. Future plans are to develop a device essentially for laboratory use that will discriminate between the effect of tire inflation pressure and tire speed as they relate to the hydrodynamic pressure that forces the water into the pavement.

SUMMARY

The laboratory methods discussed are only those that are truly for laboratory testing, except the machine used by California which may be used in both the field and laboratory. Other dual-purpose machines such as the British portable tester are not reviewed. There are also laboratory machines being used for testing of floor coverings that may have usefulness to the testing of pavement surfaces but such usefulness has not as yet been demonstrated. Undoubtedly other machines exist (for example, the one used by Jimenez of the University of Arizona, that is similar to Purdue's) which have not been developed sufficiently for reporting and are consequently unknown to the author.

Probably the greatest need in pre-evaluation of pavement mixtures is the correlation of laboratory results with field performance over a period of time.

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Factors Affecting Skid Resistance and Safety of Concrete Pavements

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The paper discusses the role of the tire and the pavement in reducing motor vehicle accidents resulting from skidding. Consideration is given to the interaction between the tire rubber characteristics of adhesion and hysteresis and the pavement surface characteristics of fine and coarse texture. Data are presented to show the importance of selecting wear-resistant fine aggregate, obtaining a proper mix design, and choosing a finishing method that will produce the desired texture depth. In addition procedures are described for restoring skid resistance to pavements that have become slippery.

•EACH year the highways are being used by more vehicles traveling at increasing speeds. The increased traffic has reduced the average distance between vehicles and this combined with increased speed has reduced alerting time to avoid obstructions. Therefore, every effort must be made to construct vehicles and highways to achieve optimum response to drivers' reactions in order to prevent accidents.

Skidding occurs in many accidents. However, it is sometimes difficult to determine whether the skid was the cause or the effect (1, 2). A report on accidents in which skidding was considered the primary cause was presented by Mills and Shelton (3). A study of these data shows that about 50 percent of the accidents on snowy or dirty pavements, 15 percent of those on clean wet surfaces, and less than 1 percent of those on dry surfaces were caused by skids. More recent data on nation-wide traffic accidents for 1966 (4) show approximately these same percentage groupings for accidents on dry, wet, and snowy-icy conditions. A survey of five states in 1964 showed that the percentage of wet pavement accidents resulting from skidding varied from zero to 23.4 percent with an average value of about 12 percent. Therefore, it is reasonable to assume, on a conservative basis, that about 15 percent of all accidents can be attributed to reduction of tire-pavement friction, and efforts must be made to overcome this fault.

This report is concerned with the role of highway design in the safety program and, in particular, the development of skid-resistant surfaces on concrete pavements to obtain desirable tire-pavement friction forces assuring maximum vehicular control.

New field and laboratory test data are reported together with pertinent findings available in the literature. Some factors affecting the skid resistance and safety of concrete pavements are examined as a part of the overall relationships between vehicles, traffic, and pavement.

SCOPE AND DEFINITIONS

This investigation is directed toward a study of the skid resistance of pavements as an aid to reducing accident frequency and severity. Of particular concern are methods of obtaining and preserving good skid resistance on concrete pavements.

Terms associated with skidding are sliding, slipping, and cornering. Kummer and Meyer (5) suggested subcategories for skidding to indicate the mode. Specifically, skidding means a sliding motion with locked wheels, and when evaluated with a skid trailer produces a skid number, often loosely called coefficient of friction. If the wheels are not completely locked, but continue to rotate while sliding, the mode is called slipping and the measure of slip resistance is a slip number. Finally, resistance to a cornering skid is cornering resistance, also called cornering force.

Of recent concern is the phenomenon of hydroplaning. When a motor vehicle moves out of control because the tire-surface friction forces provide insufficient restraint, the vehicle is either skidding or hydroplaning. If the tire maintains contact with a firm surface, the movement is generally defined as skidding. If water floats the tire at the prevailing speeds, the movement is called hydroplaning, or in some articles, aquaplaning. Hydroplaning may occur with or without locked wheels.

Moore (6) discussed two conditions of hydroplaning. The velocity that causes hydroplaning with locked wheels is less than that for rotating wheels, and is called the lower limit or sliding limit. The velocity at which an accelerating vehicle will drift out of control on a water film is the upper hydroplaning limit. If brakes are applied during upper limit hydroplaning, no control will be established until the speed has fallen through the lower limit.

MEASUREMENT OF TIRE-PAVEMENT FRICTION

Many methods and devices have been used to measure skid resistance or pavement wear. These may be grouped into (a) vehicle stopping distance tests, (b) drag of a locked wheel trailer, (c) energy loss of a pendulum, and (d) deceleration of a rotating wheel—and also variations of these principles.

Stopping distance is a measure of the distance required to bring a vehicle to a stop from a specified speed. In the test, a driver attains the desired speed, locks the brakes, and slides to a stop. The friction coefficient or skid number (skid number is the friction coefficient times 100; this term is used in the paper interchangeably with friction coefficient) can be computed from the familiar equation,

$$f = \frac{V^2}{30\,d} \tag{1}$$

where V is vehicle speed in miles per hour at initiation of the skid, and d is the stopping distance in feet.

Devices

There are a number of locked wheel trailer designs. In all cases, a towing vehicle is driven at the desired speed and water is pumped onto the pavement ahead of the trailer tires when one or both of the trailer wheels are locked. The output of either a torque or a load measuring device is usually traced on an electronic recorder during testing. The stylus displacement is interpreted in terms of the coefficient of friction by reference to a calibration curve.

The British Portable Tester (7, 8, 9) consists of a pendulum with a 1- by 3-in. rubber insert at the end that is set to slide for a distance of 5 in. over the specimen surface. Then for predetermined machine constants, the friction coefficient, f, is approximately

$$f = 0.178\ h$$

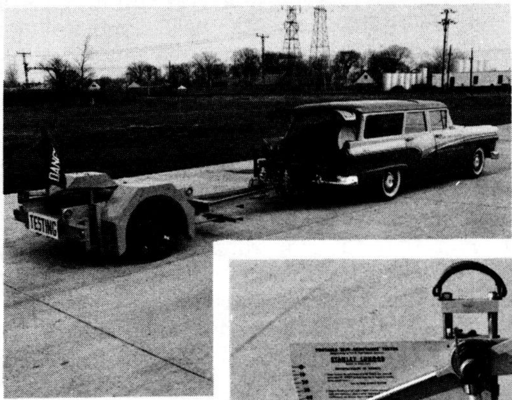
where h is the vertical distance of the scale reading below the zero, measured in inches.

A device used to evaluate pavement wear consists of a spinning automotive tire and a means of measuring power requirements to drive the tire against the test surface. Whitehurst and Goodwin (10) have described such a device, and a modified machine using this method was described by Balmer (11).

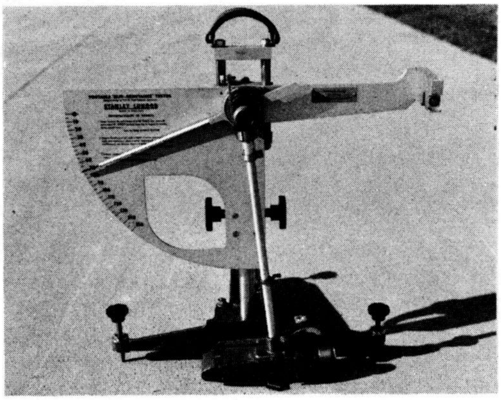
The PCA skid trailer, British pendulum tester, and a PCA device for evaluating pavement wear are shown in Figure 1.

Correlation

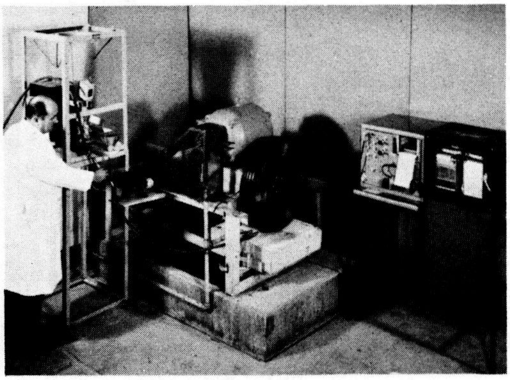
Correlation studies have been made so that values of skid resistance reported by various agencies would be meaningful. A correlation study (12) of a number of skid



◄ PCA
Skid Trailer



British ►
Portable Tester



◄ PCA Laboratory
Test Equipment

Figure 1. Typical devices for measurement of skid resistance (11).

resistance measuring machines was made at Tappahannock, Va., in 1962. Friction coefficients as determined by eight skid trailers at 50 mph on five surfaces are compared in Figure 2. These were correlated with stopping distances, the British portable tester, the National Crushed Stone Association bicycle wheel and the PDH dragtester. Considerable variation in results is evident among the trailers and similar discrepancies exist among the other devices. Friction coefficients or skid-resistance values must be qualified at present by indicating the test device and method.

FACTORS AFFECTING TIRE-PAVEMENT FRICTION

A motorist operating his vehicle can speed up, turn, or slow down. The forces at his disposal for implementing these changes are applied parallel to the pavement by means of a friction contact area between the tire and the road. Sufficient friction must develop to permit complete control while accelerating and cornering. The tire also provides a treaded surface which helps minimize the probability of hydroplaning. The

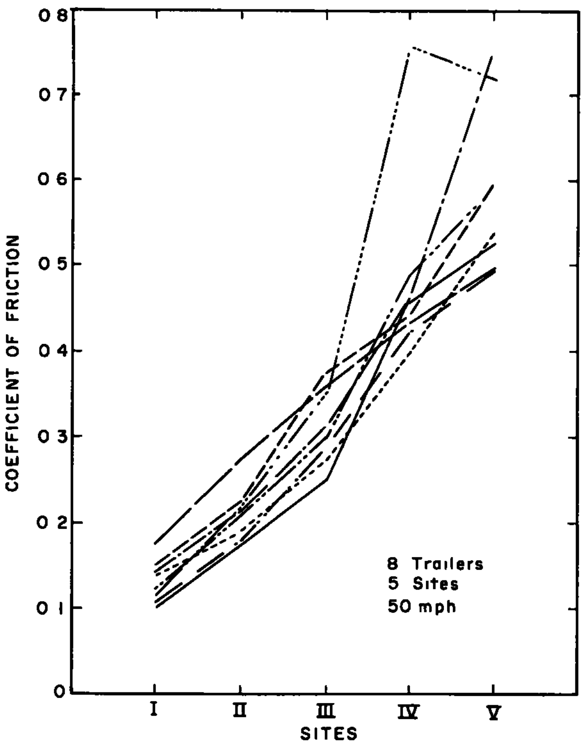


Figure 2. Comparison of trailer test results (12).

role of the pavement surface in skid resistance is to transfer necessary frictional characteristics to the tire and to provide a texture that aids drainage. Thus, frictional forces are functions of both the tire and the pavement surface, and each design should complement the other.

TIRES

Skid resistance as influenced by the tire is discussed in terms of rubber characteristics and effects of tread composition and design.

Rubber Characteristics

Adhesion and hysteresis, the two principal components of rubber friction, are indicated in Figure 3, which shows a portion of a tire tread sliding over a rough-textured pavement surface. The adhesion component is a function of the shear forces developed at the tire-pavement interface, whereas the hysteresis component is a function of the energy losses within the rubber as it is deformed by the textured pavement surface. The friction coefficient is the summation of the two components. Adhesion is of primary importance on dry pavements but assumes a lesser role on wet pavements as hysteresis becomes dominant. Maximum values of adhesion and the critical speeds at which they occur are dependent upon the elastic and damping properties of the rubber. On wet pavements, the adhesion coefficient is low and changes only slightly with increased sliding speed. The hysteresis coefficient increases with speed, with the internal damping properties of the rubber, and is practically pressure independent with little sensitivity to contamination and lubrication.

The effect of temperature (13) on the friction components of adhesion and hysteresis is influenced by the type of rubber, sliding speed, and characteristics of the surface. In general hysteresis will decrease at higher temperatures, whereas adhesion is more sensitive to speed changes and may either increase, decrease, or remain unaffected.

The resultant summation of adhesion and hysteresis comprises the coefficient of friction, and the effect of this summation for a locked wheel test on a wet pavement is shown in Figure 4 (14). The coefficient of friction decreased from 0.85 to 0.45 as sliding speed increased from 10 to 50 mph.

Tread Composition and Design

The selection of the rubber composition of tires is influenced by the tire use. A soft rubber with good friction components does not usually have sufficient tread life on highways to make this composition economically practical. Therefore, the ability to develop good friction in a harder, more durable rubber is obtained by an appropriate tread pattern.

Marick (15) has discussed the role of ribs, grooves, and slots in tire design. He showed that the tire skid number on a smooth pavement was doubled as the number of ribs was increased from 0 to 6. Most of the increase occurred with the addition of the first three ribs, and increasing the number from 6 to 12 did not result in a further significant increase.

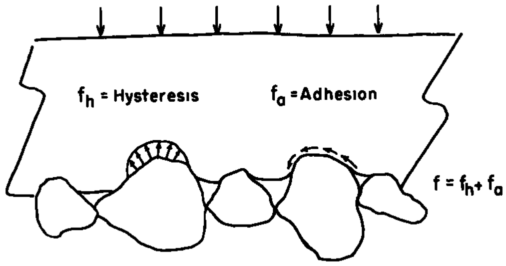


Figure 3. Friction components (13).

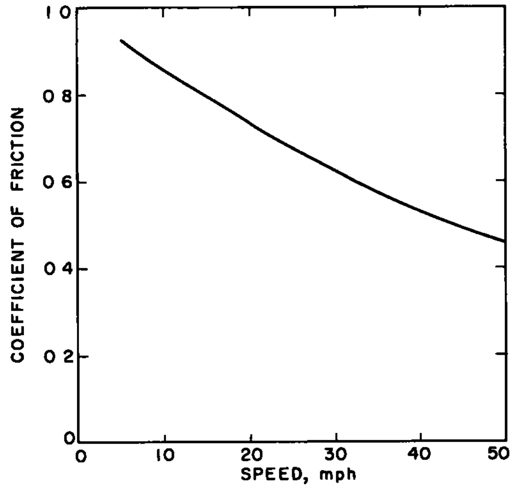


Figure 4. Effect of speed on skid resistance (14)—wet test.

The cutting of slots produces more tread edges and provides a wiping action as well as permitting venting between the ribs. Depending on tread depth, slotting increases the skid number above that of a plain rib design. Figure 5 illustrates how a tire, as it loses tread due to wear, approaches the performance of a plain rib design and eventually that of a smooth tire.

Tread design is not significant on dry pavement surfaces as smooth tires on such surfaces produce good slip or skid resistance. However, when surfaces are tested wet, tire tread gains in importance. This is demonstrated by the data of Goodwin and Whitehurst (14) shown in Figure 6. The friction coefficient values reduced more rapidly for the smooth tire than the treaded tire as speed increased, and at 40 mph the smooth tire would have required twice the stopping distance. Thus it is apparent that tire tread is significant, and that even on a highly skid-resistant surface a driver with smooth tires is a potential hazard.

The type and depth of tire tread as well as the height of pavement surface asperities are important in reducing the dangers of hydroplaning. It is generally conceded that hydroplaning can occur on a film of water if the pavement surface is smooth. For example, bald tires may hydroplane on a smooth pavement surface with a water film 0.1 in. thick, whereas greater water depths are required for treaded tires. Horne and

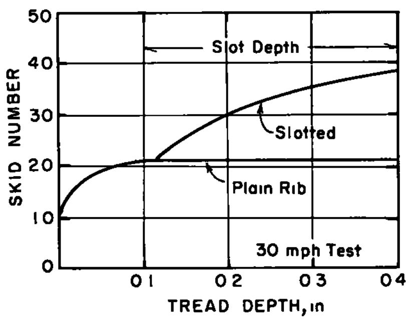


Figure 5. Skid-resistance reduction due to tread wear (15)—wet test.

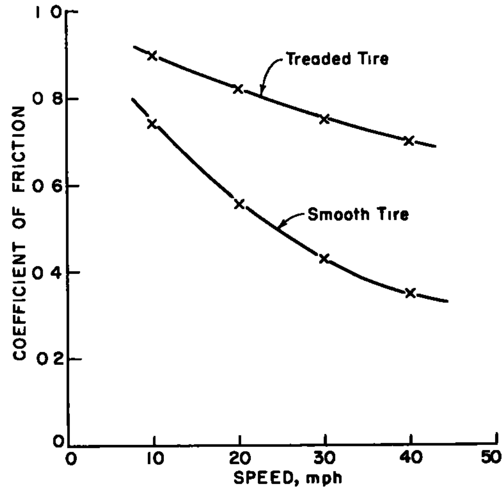


Figure 6. Effect of tire wire on skid resistance (14)—wet test.

Dreher (16) showed that for smooth or rib tread tires, where the fluid depth exceeded the depth of the tread, hydroplaning speed was given by the equation:

$$\text{Hydroplaning speed (mph)} = 10.4 \sqrt{\text{tire inflation pressure (psi)}}$$

Based on this equation, hydroplaning speed for the typical passenger vehicle could be reached at 50 to 60 mph.

The effect of water depth on friction coefficients for different speeds is shown by the data of Figure 7, reported by Trant (17). The tires had rib treads and although hydroplaning speed was not reached, there was a large decrease in the friction coefficient as water depth was increased from 0.05 to 0.30 in.

THE PAVEMENT

To obtain good tire-pavement friction values, the pavement surface must develop good hysteresis and adhesive forces in the tire rubber. In addition, permanence of the skid-resistant properties of a pavement surface is significant. Many surfaces are skid resistant when first placed but lose effectiveness due to wear. The contribution of the pavement to skid resistance is considered by defining: (a) the role and importance of texture, (b) the value of proper mix design, (c) procedures for selecting aggregates to minimize wear, and (d) methods of obtaining a suitable texture during concrete placement.

Texture

Asperities distributed over a surface form a texture. The nature of the asperities determines the texture classification, i.e., it may be coarse, fine, sharp, dense, etc. It is generally agreed that a pavement surface should have a texture consisting of both fine and coarse asperities. The fine asperities impart the adhesion component in the tire-pavement interaction, whereas the coarse asperities have the dual role of imparting the hysteresis component and providing drainage channels for water.

Texture on a bituminous pavement is created by distributing aggregate particles of various sizes in the mix and then rolling the mixture to form the surface. Thus, both coarse and fine aggregates are generally exposed at the surface. In a concrete pavement, the fine texture results from the sand-cement mortar layer and the coarse texture is formed by the finishing operations while the concrete is still plastic. Thus for concrete, the coarse texture is the ridges of sculptured mortar left by finishing marks; therefore, the coarse aggregate seldom functions as a portion of the surface. This explains why there appears to be a contradiction when a concrete engineer says permanence of skid resistance is primarily controlled by the fine aggregate and the bituminous engineer says the coarse aggregate is the major factor.

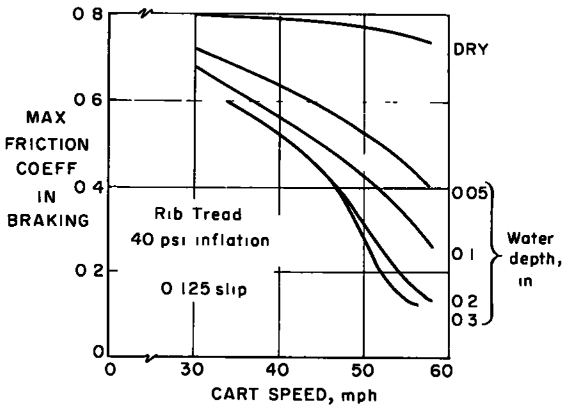


Figure 7. NACA friction-cart trough test (17).

The amount of friction developed by a surface with fine asperities has been related to the "degree of texture" as measured by various test procedures. Some procedures use techniques to obtain a cast reproduction in a plaster or resin, an ink print, or a stereophotographic view of the surface. For example, a "foil-piercing" technique was used by Gillespie (18). Data are shown in Figure 8. Attempts to corroborate these data made by the PCA on new concrete surfaces using a 5-lb weight and 2-in. drop gave impact punctures ranging between 40 to 200 per sq in. for friction coefficients varying from 0.55 to 0.88. These data, although showing considerable scatter, were in fair agreement with

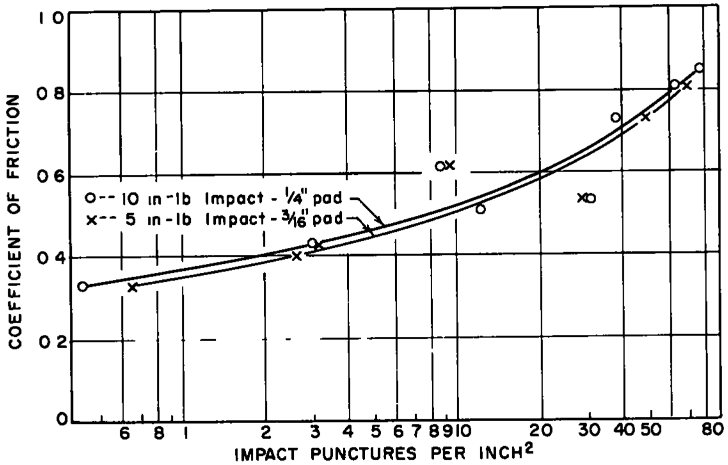


Figure 8. Coefficient of friction (35 mph) vs impact punctures.

the upper portion of the curve (Fig. 8). However on older pavements, this correlation did not exist. For example, some concrete pavements 12 to 15 years old showed no foil punctures, but when tested wet at 40 mph gave friction coefficients ranging from 0.51 to 0.62. Gillespie also reported higher than anticipated skid numbers on concrete surfaces with limited numbers of foil punctures. Thus it appears that this test procedure is not a sufficient criterion for predicting friction coefficients on concrete surfaces. This is not surprising as experience gained during 10 years of field testing with a skid trailer has shown that visual examination and sense of touch are not substitutes for a skid test.

Road contamination resulting from "traffic film" can, when wet, reduce significantly the skid number of a pavement that has fine asperties only. For example, Kummer (5) reports a rise in wet skid number from 11 to 20 for tests made just before and after a rain preceded by a long dry period. Wet skid trailer tests were made on concrete pavements by the PCA following a 0.6-in. rain that was preceded by 20 days with no measurable precipitation. The data showed that 40-mph skid numbers, obtained after the rain, had increased by a greater percentage on pavements that had the lower values prior to the rain. In general, the skid numbers obtained after the rain increased from 6 to 18 percent, with the 18 percent increase occurring on a pavement with a prior to rain value of 40, while only a 6 percent increase was observed for the pavement with an initial value of 60.

Paint is another type of contamination that reduces the skid numbers. Tests obtained by the PCA skid trailer at the Atlanta airport showed that skid numbers in painted areas were reduced from 61 to 29. It is suggested that airport areas requiring painted markings be constructed with colored concrete or with colored aggregates placed in the surface of the concrete.

Large asperities sometimes referred to as macroscopic roughness provide drainage areas under the tires. Macroscopic roughness on a concrete pavement is formed by the mortar and fine aggregates as the surface is sculptured by the finishing operations. Macro-roughness also influences the volume of tire rubber being deformed (hysteresis) and serves to increase the frictional characteristics of the pavement at greater speeds. This action is particularly valuable where conditions are conducive to hydroplaning. Such conditions occur more readily on airport pavements where the combination of relatively slow drainage of large areas and high landing speeds contribute to the problem. On highways, drainage is more readily controlled by the slope of the pavement. Slopes presently used on Interstate highways vary from about 2.25 to 5.75 in. for a 24-ft width. Because concrete pavements retain their shape, these magnitudes of slope generally provide adequate drainage.

Measurements have been made of large-scale asperity roughness using the "sand-patch" (19), "grease-smear" (20), and "drainage meter" (21) methods. Data (Fig. 9) were obtained by Sabey (7) using the sand-patch method on concrete pavements. The braking force coefficient decreased about 15 units as testing speed was increased from 30 to 80 mph. The percentage decrease varied with texture depth and it was concluded that a minimum depth of 0.025 in. was required if the decrease was to be held to 25 percent.

Data (Fig. 10) reported by Poeblikh (22) are shown as a plot of coefficient of friction versus asperity height. The friction value of both the dry and the wet surfaces increased as asperity height increased. For the wet surfaces, the coefficient was more than doubled as asperity height increased from 0.05 mm (0.002 in.) to 4 mm (0.16 in.). However, it should be noted that about 75 percent of the increase was obtained at an asperity height of 2 mm (0.078 in.).

Measurements of texture made by the PCA using the sand-patch method showed asperity heights ranging from 0.014 to 0.075 in. as the method of finish varied from a wood float to a wire brush. More complete data will be presented later when finishing techniques are discussed, but it is apparent that the texture that can be obtained by finishing will provide adequate drainage under the tires. Asperity depths of 0.075 in., although adequate, are not limiting values as even greater depths have been obtained by distributing lightweight aggregates on the surface of the plastic concrete. However it should be recognized that a pavement surface can become so rough that the skid resistance will decrease and operating cost for tires, gasoline, and vehicle maintenance will increase. In addition, rough textures can accumulate rubber deposits and cause lower skid resistance. Wet skid trailer tests made by the PCA at 40 mph on the St. Louis airport showed that rubber deposits lowered the skid number from 60 to 35.

Concrete Mix Design

Quality concrete is a prerequisite to the retention of pavement skid resistance. An improper mix design or the addition of water to the surface of the plastic concrete can result in a pavement that has a high rate of wear. Under these conditions, the benefits of a good surface texture will be of short duration.

Data obtained by Sawyer (23) demonstrating the effect of variations in cement content and water-cement ratio on concrete wear are shown in Figure 11. For 2-in. slump concrete, after 14 days of moist curing, the wear increased as much as 130 percent as the cement content was decreased. Increased water-cement ratio accompanying a change in slump resulted in an increase of wear of 20 percent.

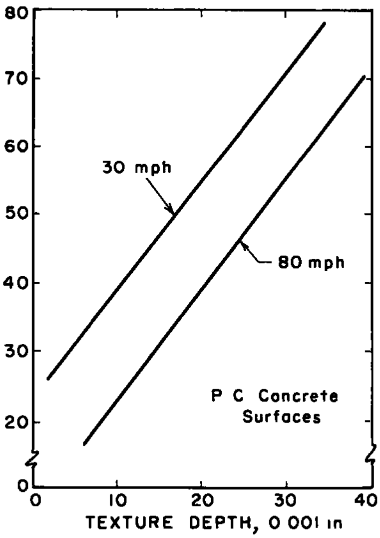


Figure 9. Trends from British data on texture depth effect on friction (7).

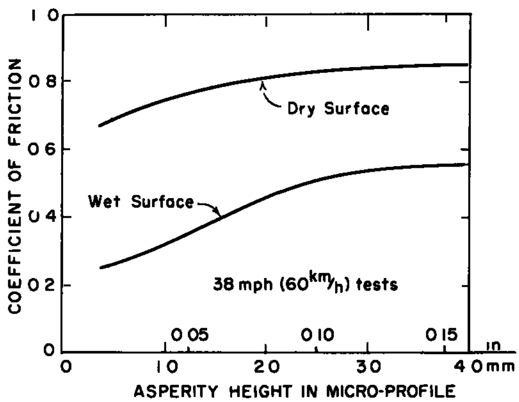


Figure 10. Data trends relating asperity height and friction coefficient (22).

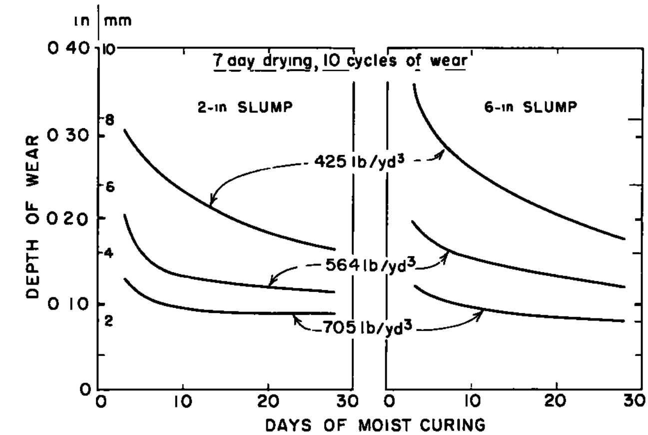


Figure 11. Effect of cement content and cure on concrete wear (23).

Data (Fig. 12) obtained at the PCA Laboratories also show the effect on wear of changes in the water-cement ratio. These tests were made using the rotating wheel equipment described previously (11). For water-cement ratios 0.47 and 0.51, wear increased by about 10 percent. The fine aggregate used in the PCA test contained about 40 percent siliceous material and, and will be shown later, aggregates of this type are less susceptible to wear. Tests were also made to study the effect on wear of changes in the cement content. Data for cement contents of 376 (4.0 bags) and 517 lb (5.5 bags) cement/cu yd were in good agreement with Sawyer's results.

To study the effect on wear of adding water to the concrete surface during finishing operations, duplicate specimens were cast at a water-cement ratio of 0.47, and 25 cu cm of water were sprinkled on the 4-sq ft surface area of the plastic concrete. Wear tests on these specimens showed that initial power requirements needed to drive the rotating wheel were decreased by about 35 percent from the values on specimens without added water, and the final wear values averaged 6. 2 instead of 6. 7.

Results of tests with varied sand content showed that the wear resistance was increased as the percent of sand was increased from 34 to 42 percent. This was probably due to a combination of a larger surface area of wear-resistant fine aggregate at the tire-pavement interface and also an increase in the cement content to maintain a constant water-cement ratio and slump when the percentage of sand was increased. However, it is suggested that PCA recommended procedures (24) be used to determine mix proportions. The amount of sand should be based on the most economical combination of available aggregates that will produce the necessary workability in the fresh concrete and the required qualities in the hardened concrete.

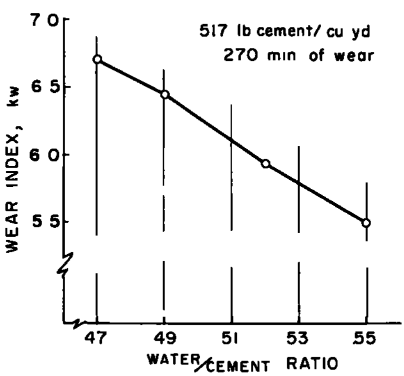


Figure 12. Effect on w/c ratio on wear.

Wear Resistance and Fine Aggregates

Concrete pavements constructed with numerous aggregate types have retained good skid resistance for many years. The data in Fig. 13 were obtained from a skid trailer survey by Barboza (25) that included 177 concrete surfaces located along approximately 6,500 miles of major highway routes in the United States. Calculations for wear index were based on age of pavement surface, average total daily traffic count, and a correction factor for traffic distribution within the lane. There was a fair degree of correlation between

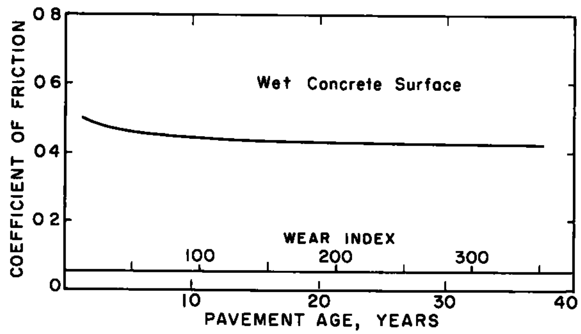


Figure 13. Effect of wear on friction coefficient (25).

wear index and age; therefore, both of these values are shown on the abscissa. The statistical best fit curve shows an average initial friction coefficient of about 0.5 and indicates that the major portion of the wear occurs in the first four or five years of service, with little further reduction as traffic life was extended to 35 years. Many other references can be cited to show similar trends of good performance. However all pavement surfaces wear and some pavements constructed with unsatisfactory materials wear so rapidly that the surfaces may become slippery after only a few years of service. Thus, the purpose of this section is to discuss methods of evaluating wear and to suggest a test procedure for selecting aggregates that will give good performance.

Scholer (26), Maclean (27), Shupe (28) and Whitehurst (29) have investigated the wear of aggregates and concrete. These studies have shown that the wear and hence the skid resistance of pavements depends principally on the mineral composition and hardness of the aggregates.

The equipment (Fig. 1) used by the PCA for evaluating wear of aggregates was described by Balmer (11). In the test, water is fed continuously to an ASTM tire held against a concrete specimen with a normal force of 600 lb while the tire is rotated at a constant speed of 250 rpm (20 mph). After 75 min of wear, a second test phase begins during which a fine Ottawa sand is blown onto the specimen with the stream of water passing between the rotating tire and the concrete. This abrades the specimen and accelerates wear. After two hours of sand abrasion, the test is continued for 75 min without the addition of sand to evaluate the worn pavement. The power in kilowatts required to rotate the wheel against the specimen is referred to as the "wear index" and is considered a comparative measure of the skid resistance.

Early in the PCA investigation, it was determined that the composition of the fine aggregate was the major factor controlling the wear of concrete pavements. Wear tests were made with fine aggregate samples obtained from 20 different sources. As these aggregates had been used to construct concrete pavements, the wear index could be correlated with in-service pavement performance. In addition, the wear index was compared with results from an acid insoluble residue laboratory test pioneered by Gray and Renninger (30).

In the acid insoluble residue test, a representative sample of 2 lb of the fine aggregate is treated with a 6N solution of hydrochloric acid. The acid dissolves the carbonates and leaves the silt, clay, and siliceous material as a residue. It is essential to use an adequate amount of acid to be certain that the action continues until completion. After the action is completed, the solution is filtered and the residue is washed with water, screened on the No. 200 mesh sieve, dried, and weighed. The silt and clay are considered detrimental to the skid resistance and are subtracted from the residue. Thus the siliceous particle content is defined as the residue retained on the 200-mesh sieve expressed as a percentage of the total sample.

The composition of each fine aggregate tested is given in Table 1, together with the siliceous particle content, the laboratory wear index, and the field performance of the pavements constructed with each aggregate. The increase in wear resistance with increasing siliceous particle content suggested that beneficiation of poor aggregates could be obtained by a partial replacement of the fines with a siliceous sand. Laboratory test results that show progressive improvement of several aggregates by beneficiation are given in Table 2. Twenty-five percent replacement was satisfactory for most of the aggregates; however, the acceptable percentages of replacement depended on the siliceous particle content of the blend. Maclean and Shergold (27) have stated

TABLE 1

EFFECT OF AGGREGATE CONSTITUENTS

Fine Aggregate No.	Principal Constituents (%)	Rating of Field Performance	Wear Index (kw)	Siliceous Particle Content (%)
1	90 calcite	Poor	3.6	2
2	70 calcite, 24 dolomite	Poor	4.4	2
3	90 calcite	Poor	4.0	3
4	80 dolomite	Poor	5.6	6
5	75 dolomite	Poor	5.8	9
6	70 dolomite	Poor	5.4	13
7	60 calcite, 16 silt and clay	Poor	5.3	17
8	80 calcite, 15 quartz	Fair	6.2	19
9	65 calcite, 12 dolomite	Poor	5.9	22
10	50 calcite, 33 mica and quartz	Excellent	6.8	33
11	55 dolomite, 39 quartz, quartzite and feldspar	Excellent	6.8	39
12	55 calcite and dolomite, 40 quartz, mica and epidote	Excellent	6 7	40
13	45 calcite, 42 quartz and feldspar	Excellent	7.3	42
14	50 dolomite, 44 quartz	Excellent	7 0	44
15	45 dolomite, 45 quartz	Excellent	6.8	50
16	45 dolomite, 45 quartz	Excellent	7.2	53
17	30 graywacke, 55 quartz	Excellent	7.0	96
18	75 quartz, 17 feldspar	Excellent	7.3	97
19	72 quartz, 20 feldspar	Excellent	7.2	98
20	99 quartz	Excellent	7.5	99

that ". . . an important characteristic of rocks that remain rough is the presence of two minerals that have a considerable difference in the resistance to wear." The data presented indicate that this concept can be extended to concrete by the procedure of blending aggregates.

A plot of wear index values as a function of siliceous particle content for the basic field aggregates and laboratory combinations is shown in Figure 14. Although there was no abrupt change in the curve to indicate a separation between poor and excellent field performance, aggregates with a wear index greater than 6.2 were classified excellent. This suggests that a fine aggregate with a siliceous particle content of 25 percent or greater will provide excellent field performance.

Ottawa sand was used to study the influence of particle size on the wear or skid resistance of concrete. This material was selected because it had a greater uniformity of hardness throughout the various particle sizes than could be found in most natural sands. Mortar mixtures with variations in the maximum and minimum particle sizes were used to apply a ½-in. thick resurface to specimens tested previously. Figure 15

TABLE 2

IMPROVEMENT OF SKID RESISTANCE BY BENEFICIATION

Basic Aggregate No.	Replacement Aggregate No.	Final Blend (%) Basic	Replacement	Siliceous Particle Content (%)	Wear Index (kw)
1	17	100	0	2	3.6
		75	25	26	5.9
		50	50	49	6.5
		25	75	72	6.8
		0	100	96	7.0
2	19	100	0	2	4.4
		50	50	50	7.0
		25	75	74	7 1
		0	100	98	7 2
4	18	100	0	6	5.6
		75	25	29	6.3
		60	40	42	6.6
		50	50	52	7.0
		0	100	97	7 3
5	11	100	0	9	5.8
		50	50	24	6.3
		0	100	39	6.8
6	18	100	0	13	5.4
		75	25	34	6.5
		50	50	55	7.2
		25	75	76	7.7
		0	100	97	7.3
7	11	100	0	17	5.3
		75	25	22	6.4
		50	50	28	6.6
		25	75	34	6.7
		0	100	39	6.8
10	18	100	0	33	6.8
		75	25	49	7.2
		60	40	59	7.4
		50	50	65	7.6
		0	100	97	7 3

shows that better wear values occurred with the larger sand size particles, although it is significant that a specimen containing Ottawa sand graded to include the total range of particle sizes from the No. 20 to 200 mesh sieves gave a wear index of 7.5, which was near the optimum value of 8.2.

In comparison, recent studies by Walz (31) concluded that the mortar surface should be composed of quartz sand well graded below 3 mm (0.12 in.). He indicated also that texture grooves of about 2.5 mm (0.1 in.) width and a low water-cement ratio combined to give good skid resistance for long periods of traffic.

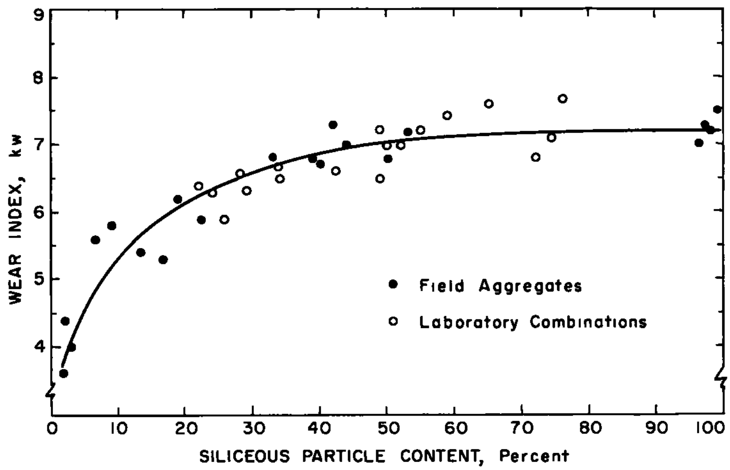


Figure 14. Effect of siliceous particle content on wear index (11).

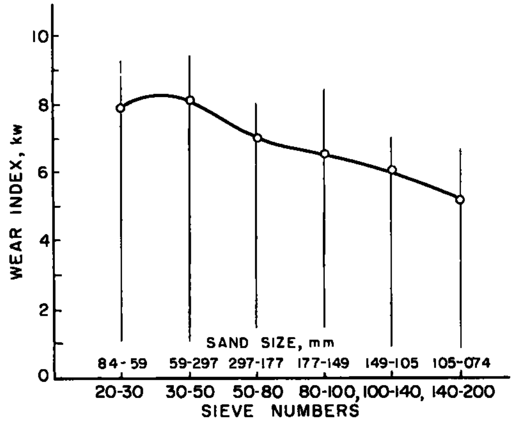


Figure 15. Effect of particle size on skid resistance (11).

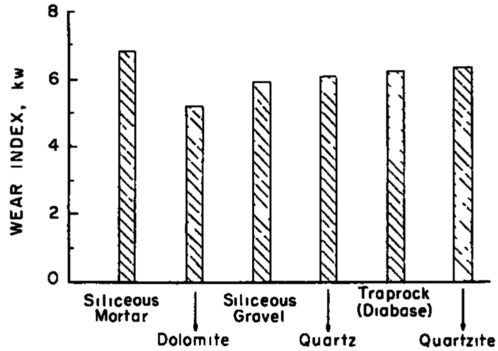


Figure 16. Evaluation of exposed coarse aggregates (11).

Concrete surfaces with exposed 1½ in. maximum size aggregates were tested with the PCA equipment to compare wear resistance with that of a conventional mortar surface. The wear index (Fig. 16) for the mortar surface made with fine aggregate No. 11 (39 percent siliceous particle content) was 6.8 as compared with values that ranged from 5.2 for the coarse exposed dolomite to 6.3 for the coarse exposed quartzite. It is conceivable that an exposed coarse aggregate with an unusually gritty texture could be more skid resistant than a conventional concrete surface; however, these tests demonstrated that a siliceous sand mortar surface was more skid resistant than many of the coarse aggregate surfaces tested.

To obtain superior friction surfaces, abrasives such as aluminum oxide and silicon carbide of 12 grit and smaller were spread over the surface of specimens and floated into the plastic concrete. The data (Fig. 17) showed the 1 lb/sq yd of aluminum oxide increased the wear resistance of a concrete with a siliceous particle content of 39 percent from 6.8 to 8.2 and increased a concrete with only 9 percent siliceous particle content from 5.8 to 7.8. Silicon carbide in the same amount was even more effective and produced values of 10.4 and 8.2. Other tests using ½ lb/sq yd of aluminum oxide yielded 7.2 and 6.4, respectively, and a like amount of silicon carbide produced 8.4 and 6.4. Abrasives in concrete may be advantageous in critical areas such as toll plazas, near busy intersections, or in areas where frequent breaking, traction, or cornering occurs.

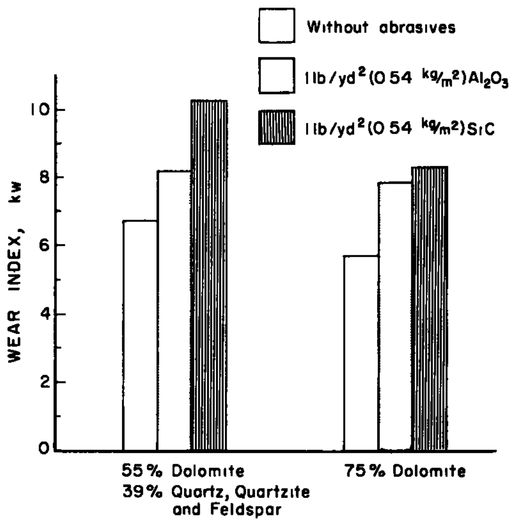


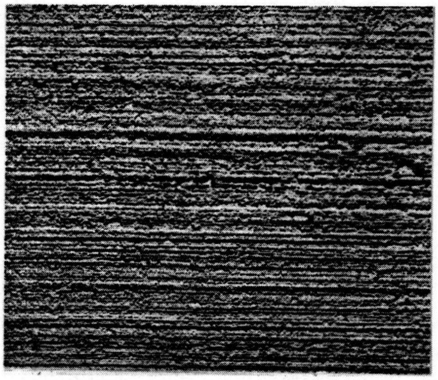
Figure 17. Use of abrasives to develop superior friction surfaces (11).

Finishing Methods and Texture

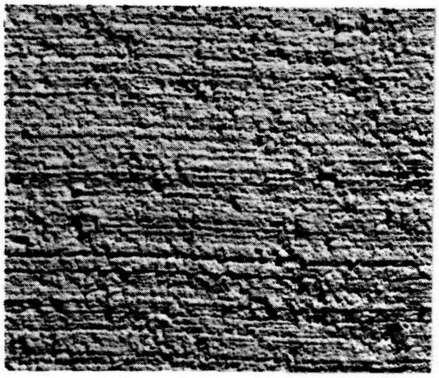
Incorporation of a properly selected fine aggregate in a quality concrete mix will provide a gritty fine-textured surface with excellent friction characteristics. Finishing operations add the coarse asperities required to aid drainage between the tires and the pavement surface and reduce the possibility of hydroplaning.

Finishing has generally been accomplished using a burlap drag, a broom, or a belt. In 1963, approximately 60 percent of the highway departments used a burlap drag, 12 percent specified either a burlap drag or a broom, 8 percent a combination of a broom and a burlap drag, 6 percent a belt, 6 percent a broom, and the remainder specified that either a belt or a burlap drag should be used.

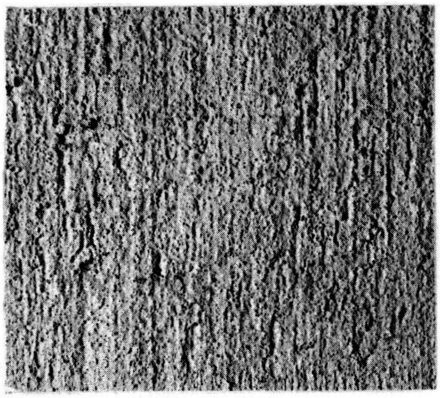
To investigate the surface texture produced by these and other finishing methods, specimens were prepared and various finishing devices were used to texture the surface



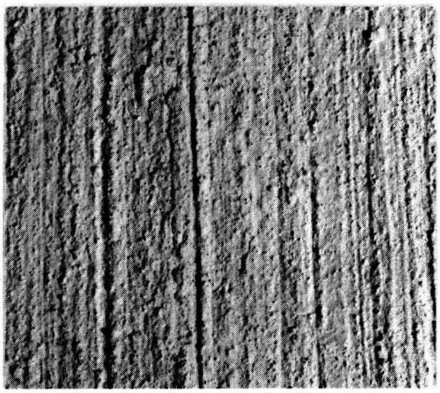
MEDIUM BROOM



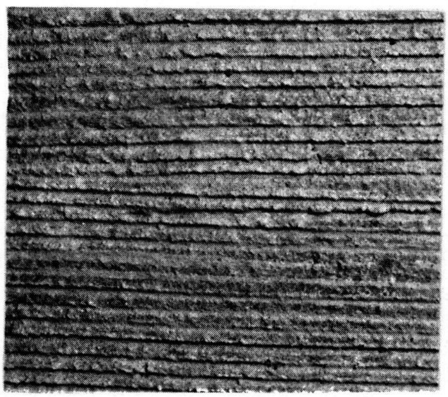
HEAVY BROOM



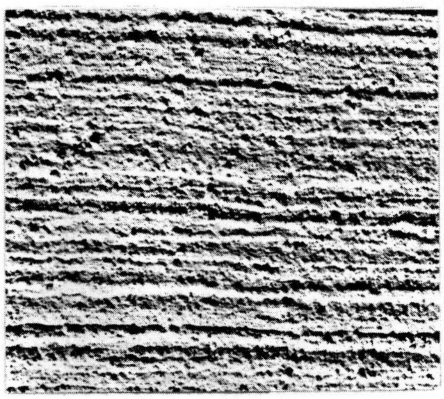
LIGHT BURLAP DRAG



HEAVY BURLAP DRAG



WIRE DRAG



FLEXIBLE WIRE BRUSH

Figure 18. Examples of surface texture.

TABLE 3

TEXTURE DEPTHS

Specimens No.	Method of Finish	Texture Depth (in.)
1	Wood float	0.014
2	Light belt	0.015
3	Light burlap drag	0.017
4	Heavy belt	0.020
5	Steel wool	0.022
6	Heavy burlap drag	0.025
7	Wallpaper brush	0.026
8	Medium paving broom	0.029
9	Door mat (cocoa matting)	0.032
10	Wire drag	0.036
11	Heavy paving broom	0.037
12	Flexible wire brush	0.051
13	Stiff wire brush	0.075

30 min after casting. Texture depths measured by the sand-patch method are listed in Table 3 and photographs of the surface texture for specimens 3, 6, 8, 10, 11, and 12 are shown in Figure 18. The largest deviation in depth of any individual specimen in a set of four from the average was only 0.003 in. This indicates that the texture depth for each method of finish was unique for the concrete mix and time interval between casting and finishing. Other data were obtained where the time interval was a variable. In general, only minor differences in texture depth were obtained with a broom for time delays between 30 min and 1½ hr or with a belt for time delays between 30 to 50 min. In contrast, texture depths obtained with a burlap drag varied considerably with the time of finishing. For example, at 20 min after casting, depths were about 25 percent larger than those listed for 30 min after casting. The time delays discussed for the laboratory tests would not be valid in the field where changing temperature, humidity, and wind velocity must be considered. However, the trends should remain the same and one way of obtaining a texture when finishing with a burlap drag would be to make three passes. The first pass should be made while there is still a slight water sheen on the surface and subsequent passes should follow without an appreciable delay between passes.

The texture depths (Table 3) obtained in the laboratory using the belt finish are less than the 0.025 in. found necessary by Sabey (7) to hold the reduction in wet skid number to 25 percent as speed increased from 30 to 80 mph. However, these small depths obtained with a belt are not compatible with the 40-mph wet skid trailer test data obtained by Barboza (25). He showed that existing pavements finished with a belt had greater initial skid numbers and retained skid resistance longer than those finished with either a burlap drag or a broom. Undoubtedly, these data represent accurately the pavements tested; however, it is the opinion of the authors that they may not be representative of what can be achieved as it is believed that only limited attention was given in the past to obtain the best possible finish for each procedure. For example, wet skid trailer tests were made by the PCA in 1967 on two sections of the same pavement, one section finished by transverse brooming and the other with the longitudinal burlap drag. On this project attention was given to obtaining a medium texture with both finishing procedures, and the average 40-mph wet skid number obtained with the burlap drag was 55 as compared to an average value of 66 for the broom finish. It is significant that skid numbers obtained on the broom finish in a direction parallel to the finish marks were about 9 percent less than those obtained transverse to the finish marks. Although these data are considered more representative of what can be done, they should be regarded also as tentative. Experimental sections should be built on new construction by each state incorporating both these and other finishing procedures.

The homemade wire drag used on specimen 10 is shown in Figure 19. Textures obtained using this drag on specimens 45 min. after casting were similar to those obtained by sawing grooves in hardened concrete. Thus a device of this type could be used to finish concrete on projects where drainage is a severe problem.

As all paving materials wear or polish, it is obvious that with other factors being equal, a pavement with a deep texture will retain skid resistance longer than a pavement with a shallow texture. But it does not follow that the texture should be as deep as possible. Instead, the finishing method selected should be compatible with the environment, speed and density of traffic, topography and layout of the pavement, and economics of vehicle operations.

METHODS FOR RESTORATION OF SKID RESISTANCE

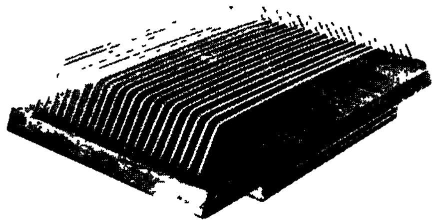


Figure 19. Wire drag.

Although there appears to be a friction coefficient that is optimum for both safety and comfort, it may be necessary to increase this value in particularly hazardous areas. Also, some aggregates polish with wear and the friction coefficient may in time be reduced to an unsafe value. Methods of increasing the skid resistance of surfaces include acid etching, mechanically abrading, sawing, and resurfacing. The choice of method depends on the economics and the end to be accomplished.

Acid Etching

An improvement in skid resistance of a concrete surface can be achieved with an acid etch. Two acids have been used for this purpose. When it is desired to dissolve the limestone and expose the silicate in the mortar, a common muriatic acid wash is applied. It is followed by a water flush when chemical action is no longer evident. If the reason for low skid resistance was a poor finish, this method is adequate; however, if the fundamental problem was soft aggregate, this treatment must be repeated frequently to assure safe skid resistance.

When it is desired to etch the silicate, a hydrofluoric acid is used. A proprietary preparation of this material is now marketed for this purpose.

Mechanical Abrading

A smooth concrete surface may be roughened mechanically with a machine with hardened steel cutters rotating on a drum, such as a Tennant machine or, as in England, the roughening may be produced by a flailing action. Depth of cut and spacing may be varied. A disadvantage of these operations is loss of the mortar layer. Also, the grooves have somewhat rounded edges. If the large aggregates are nonpolishing, these treatments may be successful. In areas where hydroplaning is a problem, drainage channels can be produced that will reduce the water film thickness and also provide escape for the water compressed beneath a tire.

Sawing

A more recent practice in mechanical beneficiation is the cutting of a series of parallel grooves with a bank of diamond or abrasive saw blades. This treatment has been recommended to aid drainage and thus reduce the tendency to hydroplane. Common practice is to groove transversely to reduce stopping distance or to groove longitudinally on curves; however, the Road Research Laboratory has data to indicate that transverse grooving is generally more effective.

Skid numbers measured by the PCA skid trailer before and after sawing a concrete pavement are given in Table 4. Three sections were sawed transversely with ⅛-in. wide by ⅛-in. deep cuts on either ¾-, 1½-, or 3-in. centers. Another section had ⅛-in. wide by ⅛-in. deep cuts sawed longitudinally on ¾-in. centers. The trailer tests were made at 40 mph with water pumped onto the dry pavement ahead of the trailer tires when both of the trailer wheels were locked. The pavement skid numbers were not significantly changed by sawing either transverse or longitudinal grooves. It should be noted that these tests were made on a concrete pavement having an average skid number of 63.

TABLE 4

SKID NUMBERS BEFORE AND AFTER SAWING

Saw Cut Spacing (in.)	Skid Number	
	Before Sawing	After Sawing
Transverse		
¾	63	60
1½	60	60
3	66	66
Longitudinal		
¾	63	63

In contrast to these results, tests reported by Horne (32) on flooded pavements representative of poor drainage indicated that both longitudinal and transverse grooves were beneficial in decreasing stopping distance. In addition, data from the California Highway Department (33) indicated that longitudinal grooving on horizontal curves greatly reduced accident rates. For example, in the 12 months before grooving 900 ft of the Santa Ana Freeway, 52 accidents occurred; in the year following, there were only 8 accidents.

From the studies described, it may be concluded that sawing grooves will be of little benefit on well-drained pavements where the maximum speed of traffic is less than approximately 40 mph. For pavements with greater traffic speeds and more surface water than that supplied by a skid trailer, sawing grooves may increase the coefficient of friction as a result of improved drainage and deformation of the tire into the grooves.

Resurfacing

Concrete overlays are well suited for improvement of skid resistance. A long-lasting surface texture meeting any requirements can be obtained easily by the final finishing operation. A number of airfields have been resurfaced with bonded concrete and performance was reported by Gillette (34). Thick concrete resurfacing without bond has been laid on both concrete and asphalt. Westall (35) reported performance of this construction on airfields in five states. An ACI Committee report (36) presents essentials of overlay design for both bonded thin and unbonded thick resurfacings.

The development of techniques for successful concrete resurfacing has been aided by laboratory studies at the PCA (37, 38). These studies indicated that before resurfacing, the old pavement should be properly prepared to obtain good bond between the new and the old concrete. Any unsound concrete and foreign matter such as oil, asphalt, or soil should be removed. The old concrete should be acid etched or scarified and cleaned. Acid etching is a desirable, economical method of surface preparation for uncontaminated sound concrete, but if the concrete is scaled, scarification is advisable. Heating the coatings of oil and grease by a high-temperature burner immediately followed by scraping has been found to be expedient for heavy contamination. In some cases, oil and grease can be removed by washing and vigorous brushing with strong detergents such as a solution of sodium metasilicate with a resin soap or a solution of trisodium phosphate. After these treatments, the concrete should be thoroughly flushed with water and cleaned. Then a thin layer of mortar or neat cement should be intimately brushed into the clean surface prior to placement of the resurface layer.

Another procedure that has been used to a limited extent is to resurface the pavement with an epoxy resin and abrasives. The surface preparation of concrete for an overlay of a resin is similar to the preparation for resurfacing with concrete, except that no mortar or neat cement layer is placed on the old concrete as a bonding medium. Ordinarily 2.5 to 3 lb/sq yd of epoxy resin (1.4 to 1.6 kg/m^2) is sufficient for surfacing.

In a PCA study (11), abrasives such as traprock, granite, expanded slag, and expanded shale were applied at a rate of 1 lb/sq yd to the epoxy resin. The sizes of the abrasives were between the No. 8 and 16 mesh (2.38 to 1.19 mm) sieves. Test results indicated that although initial skid resistance values were excellent, the epoxy resin due to moisture migration in a slab on ground did not provide a long-wearing surface.

SUMMARY

Skid resistance is an important consideration in safe vehicle operation. From a study of accident reports, it was estimated that about 15 percent of all accidents can be attributed to reduction of tire-pavement friction.

A study of methods of measuring tire-pavement friction indicated that friction coefficients or skid resistance values vary depending on the test method, and variation within methods makes it difficult to establish standard skid numbers.

Skid resistance as influenced by the tire is a function of its rubber characteristics and tread design. Rubber friction is composed principally of adhesion and hysteresis and varies mainly with the conditions of the pavement surface, sliding speed, and rubber temperature. The type and depth of tire tread are important in obtaining good skid resistance on wet pavements and in reducing the dangers of hydroplaning.

To obtain good skid resistance, a concrete pavement surface should have a texture consisting of both fine and coarse asperities. The fine asperities formed by the paste-coated fine aggregate in the mix impart the adhesion component in the tire-pavement interaction whereas, the coarse asperities formed by the sculptured surface of the concrete during finishing operations have the dual role of imparting the hysteresis component and providing drainage channels for water.

Finishing of a concrete pavement has generally been accomplished using a burlap drag, a broom, or a belt. Results of specimens finished in the laboratory indicated that texture depths obtained with a burlap drag varied considerably with the time of finishing compared with finishes obtained with a broom. It is suggested that one way of obtaining a texture when finishing with a burlap drag would be to make three passes, each at a different time interval after casting. It is obvious that with other factors being equal a pavement with a deep texture will retain skid resistance longer than a pavement with a shallow texture; however, the finishing method selected should be compatible with the environment, speed and density of traffic, topography and layout of the pavement, and economics of vehicle operations.

Quality concrete is a prerequisite to the retention of pavement skid resistance. Test data indicated that an increase in the water-cement ratio or the addition of water to the surface of plastic concrete during finishing resulted in an increase in pavement wear and caused the benefits of a good surface texture to be of short duration. A decrease in cement content also decreased wear resistance.

The skid resistance of a concrete pavement is controlled mainly by the fine aggregate in the mortar layer that is textured during finishing rather than the coarse aggregate that seldom functions as a portion of the surface. Test data indicated that a fine aggregate with a siliceous particle content of 25 percent or greater will provide good field performance.

To improve skid resistance at critical areas such as toll plazas or near busy intersections, abrasives such as aluminum oxide or silicon carbide can be spread over the surface and floated into the plastic concrete.

Methods of increasing the skid resistance of old or worn surfaces include acid etching, mechanical abrading, sawing, and resurfacing. The choice of method depends on the economics and the end to be accomplished.

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Texturing of Concrete Pavement

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Preliminary work by California on texturing of concrete pavements is described. The problem has resolved into two general areas: securing adequate texture during construction, and maintaining texture, as built, by using materials and construction practices that insure durable surface mortar.

Various texture patterns were formed into the surface of laboratory-cast slabs using a variety of prototype devices. Skid tests were performed on these slabs. A promising pattern was selected and used on short sections of three freeways. Some of the results were disappointing. A uniform texture over a large area could not be achieved because of varying mortar properties. It was also discovered that the pattern selected, when formed too deeply, caused an adverse reaction by some vehicles. Additional work is planned using other texture patterns.

Other surface treatments included broadcasting of slag and selected coarse sand particles on the surface while dragging with burlap, and brooming. Skid tests are being performed on a periodic basis, but it is too soon to draw conclusions regarding the long-term skid resistance as affected by traffic and weather.

New curing compounds and so-called surface "hardeners" were applied to short test sections of freeways in an attempt to improve mortar durability. Laboratory tests previously performed indicated that some improvement could be expected from the use of better curing compounds and hardeners. Again, these test sections have not been in service long enough to form any conclusions regarding their effectiveness.

Planned future work includes a continuation of texturing studies, a search for effective surface treatments, a study of field practices that affect surface mortar quality and texture, and additional work on grooving of older pavements to obtain or restore adequate skid resistance. Specifications will be developed as work progresses to improve character and durability of the surface texture.

•THIS paper specifically covers the activities of the California Materials and Research Laboratory on texturing and surface treatments of concrete pavements, as a part of a larger overall project on skid resistance.

The problem of poor skid resistance of some of our concrete pavements has naturally led to the examination of surface textures. The problem seems to resolve itself into two parts: getting the desired texture and maintaining it. The tools developed and the work done so far have not resulted in an ability to adequately describe the best overall texture needed. To get started, we did some preliminary work on determination of factors that affect the surface abrasion resistance of concrete.

In tackling the first part of the problem, that of getting a desired texture, we asked some specific questions. Assuming you can get any texture you want, what texture pattern is best? Is there one general pattern that is superior? Or, are there many that are adequate?

Why are present texturing procedures inadequate? First of all, we think today's high volume of traffic tends to wear down texture at a more rapid rate. Higher speed of the traffic contributes to faster wear. We also believe that some texture is lost because we now permit traffic on many new pavements very early as compared to previous practice. The quality of the surface mortar, in some cases, is apparently not good enough to withstand the high traffic density and the early use, or whatever else contributes to its disappearance. Increased wear by the use of chains and the use of sand and salt accounts for some of the loss of surface texture. Bare pavement policies in mountainous areas resulted in the loss of most as-built texture within a few months. There may be no easy solution to loss of texture from these causes in snow areas short of grooving. Textures obtained were sometimes light, a condition believed to be related to the method of forming the texture, which was not positive. Variability of texture may be due to differences of wetness and setting time of the concrete, and thickness of the plastic mortar layer.

The geometrics of texture are believed to have a great effect on durability. For example, small ridges that are formed above the basic plane of the pavement would tend to wear off faster than wider bands, formed by narrow grooves a greater distance apart.

What texture depth is necessary for long life? We can obtain a rather high coefficient of friction with a sandpaper finish, but the coefficient of friction value itself is not considered adequate to describe what we need. At least two other measures are needed: physical roughness and toughness or abrasion resistance.

We have taken the broad approach, and have encountered many different opinions regarding texturing of concrete pavement. For example, is it possible to get a consistent texture without forming it positively by molding? Can any "floating" device, such as a burlap drag or broom, form a consistent texture under varying construction conditions? How long can we maintain any texture after we get it? Is it reasonable to expect a texture to last even half as long as the pavement? Or should we plan to retexture on a periodic basis as a maintenance function? If grooved, how wide should grooves be and how far apart should they be so as not to adversely affect vehicles? Should grooves or ridges be continuous? Or, would they have a hypnotic effect on the driver? If transverse, they might cause objectional noise; how much noise is "too much?" How will this affect tire wear? Obviously, we need to know a lot more about these things. Recognizing variability in surface mortar, how can this factor be controlled during construction to provide a material that can be uniformly textured and provide the durability needed? For example, can cement or a cement-aggregate mixture be sprinkled on the surface to improve strength and therefore durability? We know that higher cement factors improve strength and durability, why not go even further than $7\frac{1}{2}$ sacks per cubic yard on the surface mortar?

Could a vacuum-type device remove excess water from the surface to produce a dense, durable mortar by lowering w/c? Or does the solution lie in the use of admixtures or hardeners on the surface, or perhaps other concreting aids such as moisture evaporation retardants? Another approach that has been suggested is that of forming no texture during initial strike-off, but return the next day or so and cut a texture in the green concrete with a machine yet to be built. Another suggestion has been the use of chips spread on the wet concrete surface and rolled in. These chips could be precoated with some material to improve bond, but there may be other materials that could be used to form nonskid textures when rolled in the fresh concrete, even "expendable" material that wears away under traffic, thereby leaving the desired texture molded in the surface.

Our approach to forming textures on pavements has been first to experiment with laboratory-cast slabs and transfer the most promising methods to the field for trial. One such promising texture developed has been used in short test sections on three freeways. This work, in some respects, has been disappointing. With small hand-operated prototypes, texture obtained has not been uniform, nor could it be formed over very large areas in the field. In some areas, the mortar cover over coarse aggregate was very thin—$\frac{1}{16}$ in. or less. Mechanical devices tend to cut through this thin layer and ride upon rocks. Other areas of the concrete hardened at varying rates which also caused a nonuniform texture. Under these conditions, rocks were dislodged and the surface torn. Timing of texturing appears to be critical. Difficulty in obtaining uniformity appears to be a good argument for positive power forming texturing devices. It may be that ultimately we will need a special texturing machine following the slipform paver which by mechanical tamping of rocks near the surface and by other vibrations and movements, would provide a neat, uniformly plastic mortar which can be extruded or formed into any texture we desire. Equipment manufacturers have been cooperative, but understandably reluctant to build machines until we can tell them precisely what we want and can define it. We believe the proper machines can be built once we know what we want.

Durability of surface mortar is an integral and perhaps the most important part of the texturing problem. Texture is lost by tire wear, and by the action of abrasives, tire chains, tire studs, salt, and freezing and thawing. We have recently completed a study of surface durability of concrete and have found results similar to the investigators. Abrasion resistance is generally proportional to strength. Therefore, any means of increasing the strength of the mortar seems worthwhile. How can this be done? This is where application of established concrete technology can be of help. Strength and abrasion resistance of mortar can, among other ways, be improved by (a) being sure materials used have the potential to produce quality mortar; (b) increasing the cement factor which is in effect lowering water-cement ratio; (c) avoiding any surface drying before curing is started, and making sure that curing is not neglected (curing is believed to be even more important than thought in the past); (d) allowing sufficient time for concrete to gain strength before subjecting it to abrasive loads; (e) and where aggressive elements will be present, like salt, using air-entrained mortar.

Some surface treatments have been demonstrated to be of some value in strengthening the mortar, but this is an after-the-fact approach and it is believed that we should first attempt to get good mortar during construction rather than rely on such treatments. They would probably be more costly than some steps we might take during construction to produce equal results. It is nice, however, to have a few tools to take care of some of these problems should other efforts fail.

All of these factors are goals of specifications, but they cannot always be met. For example, under very adverse conditions it may be necessary to use some additional water to complete finishing in order to "save" it. However, it is recognized that often, more water is used than is needed. Alternatives to not using water, such as waiting for damp weather, or covering the mortar with plastic film to prevent any drying during delay of finishing, may not be practical. In general, there are never ideal laboratory conditions in the field and some compensation may have to be provided for their absence. Perhaps the entire finishing operations currently being used need to be altered. Because of different bleeding characteristics of concrete and variable weather conditions, finishing and texturing will always be a problem, one that will require some skill and judgment in the field. One of our goals is to provide materials or procedures that are less dependent upon these factors. Things that we believe would help are more realistic specifications that recognize principles of good practice, good enforcement of these specifications, and additional requirements, if necessary, even if they make the work cost more.

In summary, specific laboratory activities undertaken so far are (a) studying factors affecting abrasion resistance; (b) using technology now available, developing specifications to improve performance; (c) developing texturing devices and trying them in the field; (d) investigating the use of admixtures and surface treatments, such as epoxy penetrants, linseed oils, and curing seals that are claimed to harden and toughen the surface; and (e) using higher grade curing compounds and different types of liquid sealants. Discussions with vendors concerning new materials have also been helpful. We have made laboratory abrasion and compression tests on cores from test sections.

Our plans are to extend and continue our activity in the field of varying finishing and curing procedures under field conditions. We will continue to investigate use of surface treatments such as chips, hardeners, special mortar applications, and monomolecular films to retard evaporation. We intend to monitor wear on established test sections by means of skid tests made periodically while taking traffic into account. We propose to extend studies of wear resistance when our laboratory tire abrasion device is available. We are attempting to develop criteria for adequate initial skid resistance.

The coefficient of friction is not by itself adequate, and we need some test to measure texture properties other than "f." Perhaps a test similar to the sand patch test used in England would be suitable, or the use of some other texture measuring device. We intend to explore the feasibility of using our abrasion test on cores from finished pavements to demonstrate adequate strength of mortar in service. We hope to develop information about the effect of a time delay in allowing traffic on pavement at various intervals after construction. We will help others develop methods of obtaining or restoring skid resistance of hardened concrete by grooving, etching, bush hammering, flail grooving, or any other method that might be proposed.

There is a lot of work yet to be done and many questions to answer. We hope that in working with contractors, materials suppliers, construction machinery manufacturers, and others concerned, we will find the proper solutions to this important highway problem.

Construction of Nonskid Pavement Surfaces

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•IN the broadest sense of the phrase, to a construction engineer, construction of nonskid pavement surfaces involves one of just two things—either building pavements initially with satisfactory skid-resistant surfaces or applying surface treatments to existing pavements to improve their skid resistance.

But, building pavements initially with satisfactory skid-resistant surfaces may be considered in terms of at least five major classifications of engineering and construction activity.

1. Application of geometric design criteria that provide for adequate profile grade, sight distance, radius of curvature, cross-slope and surface drainage.
2. Imposition of materials requirements and production controls that assure the use of hard durable aggregates with satisfactory qualities of angularity, particle size and polishing characteristics in pavement surface courses.
3. Properly designed and thoroughly blended surface-course paving mixtures that will permit full exploitation of the inherent skid-resistant qualities of individual components, e.g., asphalt-concrete mixes that will not flush and bleed in hot weather under traffic, or portland cement concrete that can develop adequate mortar strength to resist abrasion under heavy truck use, etc.
4. Execution of competent placing and finishing techniques so that the completed pavement surface possesses optimum texture characteristics in addition to proper geometric design features.
5. Timely and sufficient curing or protection of new pavement so as to assure retention of initial surface characteristics after the pavement is placed in service.

And, applying surface treatments to existing pavements to improve their skid resistance includes several additional construction activities such as grooving portland cement concrete pavement with diamond cutting equipment, applying seal coats, and resurfacing.

However, since the author is primarily concerned here with problems arising out of actual field construction practice, this presentation will be oriented to that outlook. Therefore, the sense in which the phrase "construction of nonskid pavement surfaces" is used for this article is somewhat narrower than that just outlined. This discussion will deal mainly with construction techniques utilized in placing and finishing new portland cement concrete pavement surfaces with slight excursions into techniques involved in securing nonskid asphalt-concrete pavement surfaces and in grooving existing portland cement concrete pavements to improve skid resistance.

NATURE OF THE PROBLEM

Everyone knows that a construction engineer must be constantly prepared to decide tough questions for which answers are not easy. In this matter of constructing skid-resistant pavements, though, there are many tough questions which even the most inventive, self-reliant engineer cannot now effectively answer. Perhaps the toughest question of all comes from the constructor who says, "I am ready to do anything necessary in the way of improving, changing or otherwise modifying my construction techniques—just what is it you want me to do to build a more nonskid pavement?"

What does the engineer reply to such a question? Does he specify a burlap finish, a broom finish, or some other as yet unknown type of finish? Should texturing be performed transversely, longitudinally, diagonally or in a diamond pattern, and how deep should the texture be made? Should portland cement concrete pavement finishing techniques be modified to assure a deeper grout layer at the surface, or to add specially selected particles of aggregate to the top surface course? Should the builder be required to meet a coefficient of friction specification for the completed pavement surface: a coefficient of friction requirement alone or in conjunction with some depth of texture specification? Should he also be required to satisfy some sort of a durability specification to help assure that the pavement surface will retain initial skid resistance?

These are difficult questions to answer and there are many others in a similar vein. However, reasonable answers to such questions are essential if engineers are to describe adequately and in terms of economical construction just what it is the builder must produce. The need is urgent.

REVIEW OF CONSTRUCTION PROCEDURES

Perhaps a review of construction procedures employed in California will be illustrative of some of the problems, if not the solutions. California practice has been chosen only because of the author's familiarity with it and not because it is necessarily more or less effective than that of other states.

ASPHALT-CONCRETE PAVING

Field construction procedures used in spreading and compacting asphalt-concrete pavements in California appear to have a very minor effect on the resulting skid resistance. Skid-resistance values on these pavements are generally very good but they can be either very good or very poor, depending upon whether the pavement bleeds or whether there was some other readily identifiable defect in the mix. However, because construction procedures in California appear to have such a minor effect on the resulting skid resistance of asphalt pavements, less attention will be given in this discussion to bituminous paving practices than to concrete paving practices.

It does deserve saying, though, that the favorable nonskid qualities achieved in these asphalt-concrete pavements are considered to derive principally from the physical characteristics of the aggregates and the method of mix design used.

Aggregates available in California for asphalt-concrete mixes are relatively polish resistant—limestones do not constitute a problem.

With respect to specification requirements for the aggregates, maximum particle size and combined grading is specified together with the permitted loss in the Los Angeles rattler test and the minimum percentage of crushed or naturally angular particles. Normally, the maximum particle size required is either ½ or ¾ in. (Table 1). The Hveem method of asphalt-concrete mix design results in mix with a lower asphalt content than does, for example, the Marshall method.

TABLE 1

AGGREGATE GRADING REQUIREMENTS FOR BASE COURSE AND SURFACE COURSES
(Percentage Passing)

Sieve Sizes	Base Course	Surface Courses							
		¾-In. Max.			½-In. Max.			⅜-In. Max.	No. 4 Max.
		Coarse	Medium	Fine	Coarse	Medium	Fine		
1¼ In.	100								
1 In.	95-100	100	100	100					
¾ In.	80-95	90-100	95-100	95-100	100	100	100		
½ In.					95-100	95-100	95-100	100	
⅜ In.	50-65	60-75	65-80	70-85	75-90	80-95	80-95	95-100	100
No. 4	35-50	40-55	45-60	50-65	50-67	55-72	58-75	65-85	95-100
No. 8		27-40	30-45	37-52	35-50	38-55	43-60	50-70	70-80
No. 30	12-25	12-22	15-25	18-30	15-30	18-33	20-35	28-40	35-50
No. 200	2-7	3-6	3-7	4-10	4-7	4-8	6-12	7-14	7-16

One word of caution should be noted before this brief review of asphalt-concrete paving practices is concluded. Experience indicates that surface treatments sometimes lead to loss of skid-resistance value. For example, it is not uncommon to apply what in California is called a fog seal coat, a light (0.05 to 0.10 gal/sq yd) application of asphalt emulsion, to new asphalt-concrete pavements under the initial construction contract. This practice is quite generally beneficial; however, there are conditions which can produce undesirable results. Application of such a seal late in the year, or application of an excessive amount of asphalt for the particular pavement can result in low skid-resistance values.

PORTLAND CEMENT CONCRETE PAVING

Turning to construction practices in paving with portland cement concrete, it is apparent that the situation, with respect to the importance of placing procedures on skid resistance, is significantly different than for bituminous paving practice.

For portland cement concrete, as for asphalt concrete, the aggregates must be hard and durable, but for the former they must also be of such character that it will be possible to produce workable concrete within reasonable limits of water content.

In California, aggregate requirements are established with a view primarily of structural considerations, i.e., soundness, hardness, mortar strength and shrinkage. Qualities affecting the micro- or macro-texture that can be produced on a pavement surface are not considered in establishing such requirements.

For this article, surface texture means the geometrical form of the road surface, that is, the shapes of the surface protrusions (asperities) large and small, that make contact with the tire tread, or deform it, and the channels in between them. Micro-texture refers to fine-scale asperities; macro-texture refers to the larger-scale asperities and channels in between.

With respect to specification requirements for portland cement concrete, maximum particle size and grading of the combined aggregate is specified together with cement content and limiting values of slump and total water content (Table 2).

Contractors are given the option of using either 1½ of 2½-in. maximum combined aggregate grading and they with very few exceptions choose the 1½-in. maximum option.

Slump of paving concrete is generally held within a range of 0 to 2 in. and must not exceed 3 in. In terms of Kelly ball penetration, this is a range of 0 to 4 in. and a maximum of 6 in. The free water content is also not allowed to exceed 312 lb/cu yd.

Paving concrete is now produced almost exclusively in central-mixing plants moved in and set up specifically for the purpose on each paving project. The mix is transported to the paving site in ordinary end dump trucks and currently is placed through one of three makes of slip-form pavers: Blaw-Knox, Guntert-Zimmerman, or Concrete Machinery Incorporated.

The make of slip-form paver used has some influence upon the completed pavement surface condition. Certain accessory equipment items, such as the rotating screed, which may be either part of the original equipment or contractor's additions, also have an influence on the condition of the finished pavement surface.

It is reasonable to assume that adding additional water during the

TABLE 2

GRADING LIMITS OF COMBINED AGGREGATES

Sieve Sizes	Percentage Passing		
	2½-In. Max.	1½-In. Max.	1-In. Max.
3 In.	100		
2½ In.	95-100		
2 In.	80-95	100	
1½ In.	65-87	90-100	100
1 In.	50-75	50-86	90-100
¾ In.	45-66	45-75	55-100
⅜ In.	38-55	38-55	45-75
No. 4	30-45	30-45	35-60
No. 8	23-35	23-35	27-45
No. 16	17-27	17-27	20-35
No. 30	10-17	10-17	12-25
No. 50	4-9	4-9	5-15
No. 100	1-3	1-3	1-5
No. 200	0-2	0-2	0-2

finishing operation, or mixing bleed water with the surface mortar, will have significant effect on the strength and wear resistance of the surface and thus upon durability of the macro-texture. Since rotating screeds are conventionally considered to require the addition of water to function properly, use of this equipment may result in significant loss of wear resistance, assuming of course that an acceptable finish can be obtained by other techniques that do not result in equivalent loss. This is a controversial issue and slip-form pavers both with and without rotating screeds are used.

One make of slip-form paver currently used utilizes two heavy oscillating screeds as the primary means of shaping the pavement surface. Appearance of the completed surface confirms the logical conclusion that a deeper layer of grout is produced than with pavers utilizing a conforming (extrusion) screed. Any effect this may have upon the effectiveness or durability of the completed surface texture is also controversial and use of the equipment is not restricted on this basis.

Following the slip-form paver, hand-finishers repair defects in the surface, such as tear marks and edge slump. The entire surface is then screeded with a pipe float or a diagonal wood float. Generally, three passes of these devices are sufficient to eliminate minor variations in the surface due to differential compaction, tear marks, etc.

It is quite common to see a fine spray of water being added to the surface to facilitate the action of pipe and diagonal wood floats, or the timing of the operation is such as to cover the span of the concrete bleeding period so that excess water from this source is intermixed with the surface grout. Here again, the necessity for and effect of these techniques with respect to the effectiveness and durability of the completed surface texture is controversial.

As with the addition of water at rotating screeds on certain pavers, or in connection with handwork, paving inspectors attempt constantly to restrict the use of additional water to only that lost from the surface by evaporation. Unfortunately, this is an activity in which finishers and others in the paving crew all too frequently resist and circumvent whenever possible.

The final finishing operations are to texture the surface and then apply curing compound. There is little doubt that initial skid-resistance values depend largely on effectiveness of the texturing operation. There is even less doubt that durability of the initial nonskid texture is to a very important degree dependent on effectiveness of the cure. These final finishing operations are, therefore, much more critical than the attention ordinarily given them would indicate.

Texture is routinely accomplished by use of burlap drags or the combination of an initial burlap drag and a final longitudinal brooming with nylon bristle brooms.

Curing is accomplished by spraying the fresh concrete surface with white pigmented curing compound at the rate of 150 sq ft/gal. Compliance with the specified application rate is checked by comparing quantities of compound expended with the area of pavement surface covered, either on a spot check or continuing basis. Caution must be exercised, however, because the application rate rarely if ever equals the expended rate. This, of course, is due to several factors but without doubt the major factor is wind drift. Thus, many occasions occur when the spray rate must be increased, or the area given a double application of compound in order to assure effective curing.

Timeliness in applying the compound is probably as important, if not more so, than the rate of application in achieving an effective cure and thus long-lasting nonskid surfaces. Delay in applying curing compound is a widespread and commonly overlooked shortcoming in most warm weather paving operations. It occurs because curing must follow the hand-finishing and texturing operations. Timing of the texturing operation is routinely a compromise, since concrete pavements rarely lose surface moisture uniformly. Optimum texturing, at least when burlap drags and brooms are used, occurs when the wetter spots have dried enough to reasonably hold the texture, but before the drier spots have dried too much to texture at all. However, in terms of optimum curing, this usually means that the drier areas are too dry.

A partial solution to this dilemma is to speed up and improve the effectiveness of final finishing operations. Improved equipment such as the combined pipe float-burlap drag-curing compound applicator recently making an appearance on California paving projects would appear to be an important advance in the right direction. It is suggested that an even more important advance would be the incorporation or improvement of metering screeds at the receiving end of slip-form pavers to reduce the slight variations in surface condition that are one of the primary causes of localized excessively wet spots.

Some progress in this direction is evident in the widespread use of box spreaders ahead of some pavers, or in the auger at the front end of one paver, or contractor modifications such as strike-off paddles added to other pavers.

The completed pavement may be opened to traffic as early as 10 days following concrete placement, unless Type III cement is used and an earlier opening is permitted in writing by the engineer. In that event the pavement may be opened to traffic as soon as the concrete has developed a modulus of rupture of 450 psi.

California specifications require that new pavements must have a coefficient of friction of not less than 0.30 as determined by Test Method No. Calif. 342 (see p. 119 of this publication).

The construction aspects that can be expected to affect performance of new concrete pavement surface texture would seem to be as follows:

1. Nature of the concrete aggregate.
2. Amount of mixing water used.
3. Nature and amount of any admixtures used.
4. Cement factor.
5. Nature and amount of surface manipulation in finishing.
6. Amount of additional water used during finishing.
7. Amount of bleed water mixed into surface mortar.
8. Timing of finishing operations.
9. Method and depth of texturing.
10. Orientation of texture serrations.
11. Timing of the curing operation.
12. Effectiveness of curing materials, as applied.
13. Extent of curing period before pavement is opened to traffic.
14. Amount of abuse by construction equipment in constructing appurtenant facilities.
15. Extent of bump cutting to meet smoothness requirements.

GROOVING EXISTING CONCRETE PAVEMENT

Grooving hardened portland cement concrete pavement to improve skid resistance is becoming an increasingly important construction technique. This work is done currently through the use of two different makes of concrete planing machines: the Concut Bumpcutter and the Christensen Concrete Planer. In each case, a cutting arbor of multiple diamond saw blades separated by appropriate spacers produces a series of parallel grooves in the pavement surface.

When grooving an existing pavement, California engineers have generally adopted a grooving pattern in which $\frac{1}{8}$-in. grooves spaced at $\frac{3}{4}$ in. on centers are cut at $\frac{1}{8}$-in. minimum depth. However, a recent innovation has been to utilize 0.095-in. wide blades instead of the $\frac{1}{8}$-in., that is 0.125-in., blades to cut narrower grooves. The narrower groove pattern may produce less disturbance in the control of motorcycles operated over the grooved pavement and other advantages.

The features of grooving that can be expected to affect skid resistance are as follows:

1. Width and depth of grooves.
2. Spacing of grooves.
3. Amount of lane width grooved.
4. Orientation of groove pattern, i.e., longitudinal, transverse, or other.

California pavements are grooved longitudinally on the premise that lateral resistance to skidding on curved alignment by highway vehicles is improved over that of transverse or diagonal grooving. Substantially the whole lane width is grooved, omitting only about 12 in. adjacent to each lane line or edge of pavement.

One of the newest construction developments with respect to grooving is the introduction by equipment manufacturers of vacuum devices which permit the removal of water and cutting residue concurrently with the grooving operation. This should greatly reduce the hazard to traffic created when water is allowed to flow across active lanes of pavement.

CONCLUSION

Before concluding this brief examination into the relationship of construction procedures and skid-resistant pavement surfaces, it may be well to seek the view point of the highway user. Can it be doubted that in his eyes the only thing actually being done about improving his protection from skids is what is presently being constructed into the roadways he drives on or the vehicles and tires he uses?

Engineers and scientists realize that, as in the case of building Rome, construction of the optimum in nonskid pavement surfaces cannot be accomplished in a day. However, improvement can be realized a step at a time.

Much is now being accomplished by way of enlarging the technical understanding of tire-pavement interactions and means of measurement. Hopefully many of the papers presented at this summer meeting of the Highway Research Board will contribute to the growing knowledge. Important as this may be, though, sight must not be lost of those areas of activity in which improvements can be made in the skid resistance of existing pavements and those currently being built.

Reduction of Accidents by Pavement Grooving

JOHN L BEATON, ERNEST ZUBE, and JOHN SKOG,
California Division of Highways

Providing and maintaining a skid-resistant surface on concrete pavements is discussed. Studies of the effect of grooving the pavement to reduce wet weather accidents were conducted. The objective of the studies was to determine the efficiency of serrations in raising the skid resistance, to determine resistance of a grooved pavement to wear and polish of traffic, and to determine the extent of reduction in wet weather accidents by serration of the pavement. Results show that pavement grooving parallel to the centerline will reduce the wet weather accident rate in low friction value areas of PCC pavements. Friction value is raised following grooving. Wear and polish of grooved areas appear to depend on characteristics of the pavement.

•PROVIDING and maintaining a skid-resistant surface is of primary importance to the proper performance of any highway. All types of pavement surface will eventually show some reduction in coefficient of friction values during their service life. This reduction is caused by wear and polish of traffic, especially by heavy trucks.

Several years ago California Division of Highways accident analysis showed that some sections of concrete freeways, especially on curves, were having an unusual number of accidents occurring during wet or rainy weather. After considering the use of acid treatment of the surface or the application of a coal tar-epoxy screening seal coat or some other thin organic overlay, it was decided to study the effect of grooving the pavement. The objectives of the program are as follows:

1. To determine the efficiency of serration in raising the skid resistance.
2. To determine the resistance of a grooved pavement to wear and polish of traffic.
3. To determine the extent of reduction in wet weather accidents in critical areas by serration of the pavement.

MEASUREMENT OF SKID RESISTANCE

The California skid tester used in determining the coefficient of friction of pavement surfaces has been previously described (1). The presently used test method is given in the Appendix. The tester has been calibrated with the towed trailer equipment constructed by R. A. Moyer of the University of California, Institute of Transportation (2). Previous studies by Moyer and others indicated that the skid-resistance value for any given surface approaches a low figure when the brakes are locked on a vehicle having smooth tread tires and traveling at speeds of 50 mph on a wet pavement. Therefore, in the correlation program, the coefficient of friction values obtained from Moyer's unit using locked wheels, smooth tires, wet pavement and a 50-mph speed were compared to our readings obtained under identical operating conditions.

We are presently using a value of $f = 0.25$ as the minimum requirement for indicating the need for remedial action, and a minimum of $f = 0.30$ for new PCC pavement. An active program is under way to study the adequacy of these values, especially the figure for remedial action. The program involves the use of recommended minimum friction values from other sources, and an accident frequency correlation with skid resistance of the pavement surface.

C. G. Giles (3) on the basis of a comprehensive accident analysis in England has provided a set of suggested values of skid resistance for use with the British portable

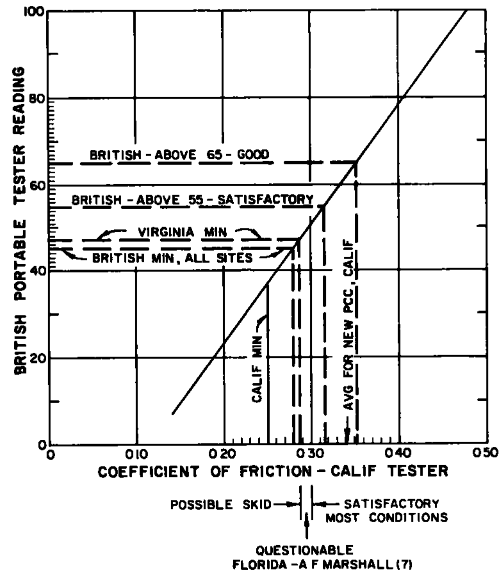


Figure 1. Correlation studies on minimum friction value for remedial action.

tester. A comprehensive correlation program was performed by us in order to obtain the relation between the California tester and the British portable tester.

On the basis of the correlation, a comparison of the recommended British values with the tentative California minimum figure is shown in Figure 1. Also shown is the Virginia minimum figure which was attained by using the correlation chart of D. C. Mahone (4) which provides an approximate correlation between the British portable tester and the Virginia test car at 40 mph.

Preliminary studies in California on a wet weather accident frequency correlation with skid resistance indicates that most single car accidents occurred on curves, the average value for the friction factor being f = 0.22. However, 28 percent of the accidents that occurred on curves were on pavements of f = 0.25-0.28 range. The maximum value attained in this study was f = 0.28.

On the basis of these results, it is concluded that the present f = 0.25 remedial action value is a minimum figure, and it appears that the value may be too low for curves of rather short radius. A better value may be f = 0.28 which is the same as the British minimum for all sites. Further studies are under way.

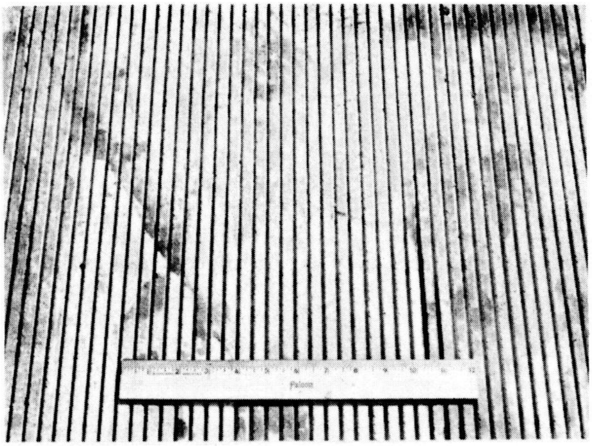
PATTERN STUDIES

Grooves may be cut in the pavement in either a longitudinal (parallel to the centerline), transverse or skewed direction. All grooving (except for a few short experimental sections) to date on state highways has been performed in a longitudinal direction. We are of the opinion that this leads to increased lateral stability, and tends to guide the vehicle through a critical curve area. This has been confirmed by studies performed in Texas (5). However, studies in England (6) indicate that grooving perpendicular to the centerline is better in this connection; further effort will be required to resolve the problem.

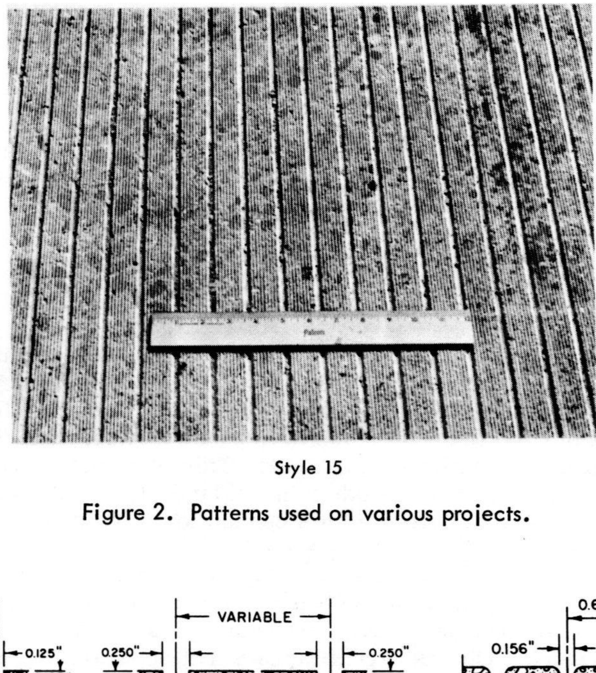
Groove patterns vary. The most common type is rectangular in form and may be varied in width and depth and distance between centers of grooves. Other types have rectangular form, but the bottom is partially rounded, and the edges at the pavement surface are also rounded. Others have a large V-cut separated by smaller V-cuts. Figure 2 shows two types of patterns.

A number of patterns have been used in our serration work to date. This was done in order to determine the increase in the friction factor, wear resistance, and possible vehicle handling problem. In all cases the grooves are all in a longitudinal direction. Figure 3 shows the patterns used on the various projects, and Table 1 gives the increase in the friction value after grooving and the change during service life. Figure 4 shows the effect of grooving on the average coefficient of friction value for the various PCC pavement projects.

In all cases the friction value is raised by pavement grooving. However, it appears that the nature of the existing concrete surface and the type of pattern affect the degree of improvement in the friction value. As an example there is a much greater improvement in the friction value for project H than on projects F and G for a $\frac{1}{8} \times \frac{1}{8}$ in. on 1 in. centers with a rectangular groove. This is also confirmed by the results from projects J and K where two different patterns are compared on two different projects.



Rectangular Grooves



Style 15

Figure 2. Patterns used on various projects.

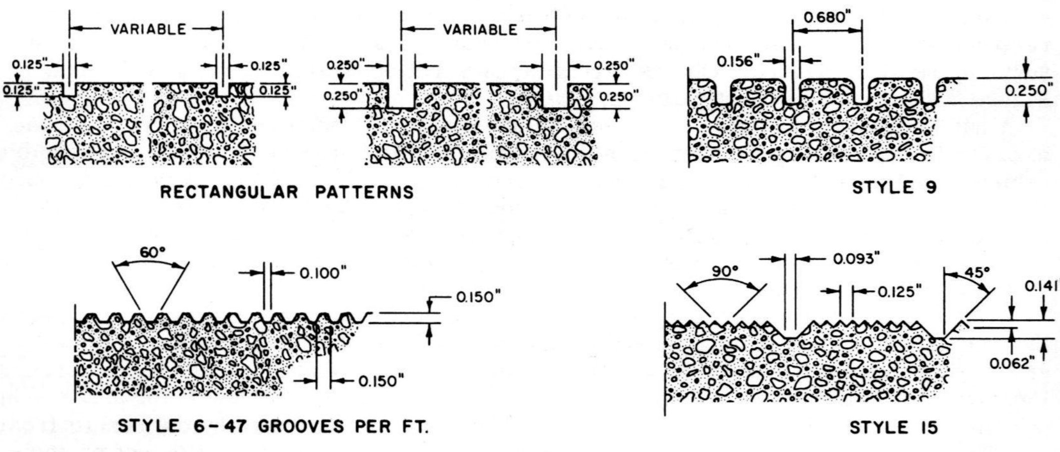


Figure 3. Grooving patterns used on various projects.

TABLE 1

CHANGE IN AVERAGE FRICTION VALUES FOLLOWING GROOVING

Project No	Pavement Type	Location	AADT/1000	Serration Pattern	Age Mos.	Avg Friction Value
A	PCC bridge deck	10-Sta-4-A	24	Rectangular grooves 1/8 in. × 1/8 in. on 3/8 in. centers	Before After 45 101	0.26 0.33 0.33 0.36
B	PCC bridge deck	04-Ala-7-Alb	80	Rectangular grooves 1/8 in. × 1/8 in. on 3/8 in. centers	Before After 41	0.26 0.32 0.28
C	PCC	06-Kern-5- PM6.94-7.47	16	Rectangular grooves 1/8 in. × 1/8 in. on 3/8 in. centers	Before After 67	0.19 0.32 0.34
D	PCC	07-Ora-5- PM23.3-23.6	45	Rectangular grooves 1/8 in. × 1/8 in. on 1/2 in. centers	Before After 17	0.25 0.35 0.30
E	PCC	07-LA-5 PM29.5-30.0	104	Rectangular grooves 1/8 in. × 1/8 in. on 3/4 in. centers	Before After 17	0.23 0.31 0.27
F	PCC	07-LA-405 PM2.1-2.6	131	Rectangular grooves 1/8 in. × 1/8 in. on 1 in. centers	Before After 17	0.20 0.24 0.22
G	PCC	07-LA-405 PM3.8-4.1	139	Rectangular grooves 1/8 in. × 1/8 in. on 1 in. centers	Before After	0.19 0.21
H	PCC	03-Pla, Nev-80 Var.	9	Rectangular grooves 1/8 in. × 1/8 in. on 1 in. centers		
H-1		E.B. lane PM42.56-42.77			Before After 12 Mo.	0.24 0.37 0.34
H-2		W.B. lane PM45.45-45.60	9	Rectangular grooves 1/8 in. × 1/8 in. on 1 in. centers	Before After 12 Mo.	0.25 0.32 0.29
H-3		W.B. lane PM5.00-5.27	9	Rectangular grooves 1/8 in. × 1/8 in. on 1 in. centers	Before After 12 Mo.	0.19 0.29 0.27
H-4		E.B. lane PM6.55-6.65	9	Rectangular grooves 1/8 in. × 1/8 in. on 1 in. centers	Before After 12 Mo.	0.15 0.30 0.25
H-5		W.B. lane PM9.01-9.19	9	Rectangular grooves 1/8 in. × 1/8 in. on 1 in. centers	Before After 12 Mo.	0.19 0.30 0.27
I	AC	07-LA-101 PM8.8-9.3	134	Rectangular grooves 1/4 in. × 1/4 in. on 1 in. centers	Before After 17	0.23 0.28 0.29
J	PCC	07-LA-14-27.89 Placerita Canyon Bridge	13	Christensen Company Style 9	Before After	0.16 0.26
				Christensen Company Style 15	Before After	0.16 0.33
K	PCC	07-Ven-101	21	Christensen Company Style 6	Before After	0.20 0.37
				Christensen Company Style 9	Before After	0.20 0.31
				Christensen Company Style 15	Before After	0.19 0.37

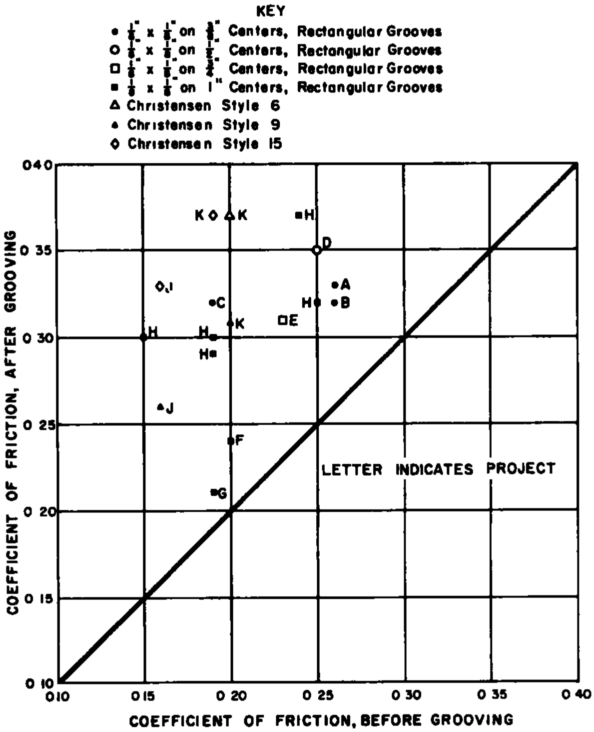


Figure 4. Effect of grooving pattern on average coefficient of friction value of PCC pavements.

The type of pattern on any specific project effects the degree of improvement. On project K, three different Christensen patterns (Table 1) were placed in consecutive 100-ft test sections in the travel lane. The original coefficient of friction values were identical, but two of the patterns produced a very high degree of improvement as compared to the third pattern.

Project I in District 07 is an aged asphalt-concrete pavement. The surface was rather dry in appearance and quite brittle. Therefore, it was decided to groove this pavement using $1/4 \times 1/4$-in. grooves on 1-in. centers. Shortly after completion several complaints were received from drivers of motorcycles and light cars. The complaints were that the vehicle tended to "track" and appeared to be caught in a manner resembling being caught in streetcar tracks. This was confirmed by highway patrolmen. On the other hand, Christensen Style 15 with V-cuts 1/4 in. wide on the Placerita Canyon Bridge provided no problems with test motorcycles driven up to 70 mph. There was some vibration up to 50 mph with Style 9, but this tended to fade out at higher speeds. Style 9 (Table 1, project J) has 3/16-in. wide rectangular grooves with rounded bottom and edges. These studies indicate that rectangular longitudinal grooves should not be wider than 1/8 in. in order to prevent possible problems from motorcycles and light passenger cars. However, V-cuts do not appear to cause problems although 1/4 in. wide at the surface.

An important characteristic of any treatment for raising the existing friction value is its resistance to wear and polish of traffic. Results of variations of friction measurements with time on various grooved projects are shown in Table 1 and Figure 5. On the majority of the projects not enough time has elapsed to draw any firm conclusions. It appears, however, that the nature of aggregate and mortar strength may influence the resistance to wear and polish of the grooved areas. However, it is interesting to note that projects A and B cover the travel lanes of heavily traveled freeways having a high percentage of trucks. All the projects shown in Figure 5 are in

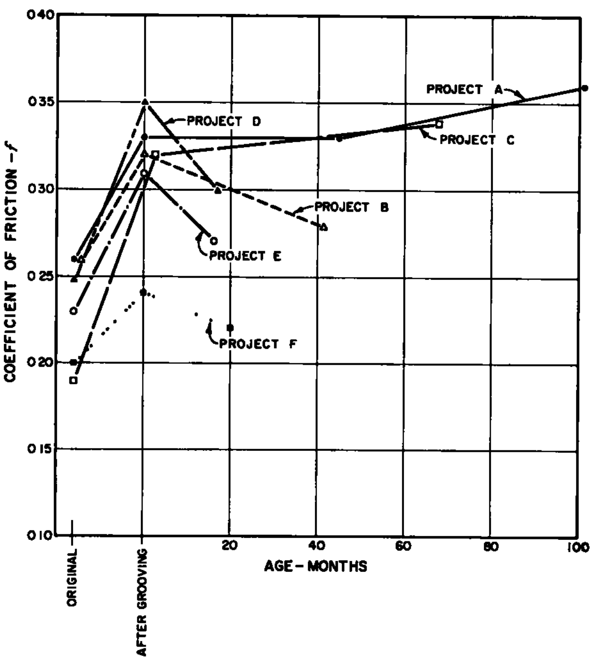


Figure 5. Change in friction values following grooving of PCC pavements.

snow-free areas. Project H (Table 1) is in a partial snow region where chains may be required. After the first winter the surface does not appear to be damaged by chain action. This project will be closely watched since project M in Table 2 has shown considerable spalling between the grooves which are on 1-in. centers. This spalling has been caused by chain action and has resulted in some complaints in regards to controllability of a car even under dry pavement conditions.

ACCIDENT STUDIES

Summaries of all of the presently available accident data are given in Tables 2, 3 and 4. Six of these locations were on urban freeways in the vicinity of Los Angeles. Accident data were also reviewed for comparison purposes on a mile of unserrated asphalt-concrete freeway (Table 4). The Los Angeles projects had one year before and after accident analysis periods. An additional project M on I-80 near the Nevada state line had a two-year period for before and after accident analysis. This freeway is rural and required longer periods to obtain meaningful data. In the case of the Los Angeles area freeways, the number of wet or rainy days was determined in both the before and after accident periods. There were 30 wet days in the before period and approximately 15 wet days in the after period. Fifteen additional wet days were accumulated from the following year and the accidents on these days were added to the after period.

Table 2 indicates that the total accidents were reduced 78 percent; of this, wet pavement accidents were almost completely eliminated (96 percent) and dry pavement accidents dropped 32 percent.

The reduction in dry weather accidents, if confirmed by further observation, appears to be significant. There is no reason to doubt that the dry friction value of these pavements was sufficiently high. In our opinion the decrease in dry weather accidents may be the result of the ability of the grooves to "track" or aid as a guide for a vehicle nearing an out-of-control condition in the curve area. Such loss of control would most commonly be caused by entering the curve at excessive speed and then rapid deceleration

TABLE 2

EFFECT ON NUMBER OF ACCIDENTS FOLLOWING GROOVING

Project No.	Location and Pvt Type	Serration Pattern	Curvature		AADT/1000	Accidents								
			Radius (ft)	Dir		Before			After			Percent Change		
						Wet	Dry	Tot	Wet	Dry	Tot.	Wet	Dry	Tot.
D	07-Ora-5 PM23.3-23.6 PCC	1/8 in. × 1/8 in on 1/2 in. centers-rectangular grooves	2000	Right	45	46	4	50	1	7	8	-98	+75	-84
E	07-LA-5 PM29 5-30 0 PCC	1/8 in × 1/8 in. on 3/4 in. centers-rectangular grooves	2000	Left	104	12	6	18	2	2	4	-83	-67	-78
L	07-LA-10 PM22.6-22 8 PCC	1/8 in. × 1/8 in. on 3/4 in centers-rectangular grooves	1020	Left	164	26	16	42	0	6	6	-100	-63	-86
F	07-LA-405 PM2 1-2.6 PCC	1/8 in × 1/8 in. on 1 in centers-rectangular grooves	Tangent		131	21	9	30	0	11	11	-100	+22	-63
G	07-LA-405 PM3 8-4 1 PCC	1/8 in × 1/8 in on 1 in centers-rectangular grooves	3000	Right	139	4	6	10	0	4	4	-100	-33	-60
M	03-Nev-80[a] PM19.8-20 2 PCC	1/8 in × 1/8 in. on 1 in. centers-rectangular grooves	1400	Left	9	5	9	14	0	6	6	-100	-33	-57
N	07-LA-101 PM8 8-9 3 AC	1/4 in × 1/4 in. on 1 in centers-rectangular grooves	2050 2052	Reversing	134	139	55	194	6	35	41	-96	-36	-79
Total						253	105	358	9	71	80	-96	-32	-78

[a]Two year before and after period, all others one year

TABLE 3

EFFECT ON ACCIDENT RATE FOLLOWING GROOVING

Project No	Pvt Type	U-Urban R-Rural	State Avg Acc Rate	Before						After					
				Wet		Dry		Total		Wet		Dry		Total	
				MVM[a]	Rate[b]	MVM	Rate	MVM	Rate	MVM	Rate	MVM	Rate	MVM	Rate
D	PCC	R	1 00	0.18	255 56	1 96	2.04	2 14	23 36	0.20	5 00	2.26	3 10	2 46	3.25
E	PCC	U	1.61	0.77	15 58	8.54	0.70	9 31	1 93	0.78	2.56	8.71	0.23	9.49	0.42
L	PCC	U	1 61	0 49	53 06	5 46	2 93	5.95	7 06	0.49	0.00	5 50	1 09	5 99	1 00
F	PCC	U	1.61	0 97	21 65	10 80	0 83	11 77	2 55	0.98	0 00	10 97	1 00	11 95	0 92
G	PCC	U	1 61	0 59	6.78	6 64	0 90	7 23	1 38	0 63	0.00	6 98	0.57	7 61	0 53
M	PCC	R	1 00	—	—	—	—	1 02	13 73	—	—	—	—	1.31	4.58
I	AC	U	1.61	2 04	68 14	22 78	2 41	24 82	7 82	2.01	2.99	22 45	1.56	24 46	1 68
Total for PCC Pvts.			1 48	3 00	36.33	33.40	1 23	37 42	4 38	3 08	0 97	34.42	0 87	38 81	1.00

[a]MVM = million vehicle-miles
[b]Rate = number of accidents - MVM

TABLE 4

COMPARISON OF NUMBER OF ACCIDENTS ON GROOVED AND CONTROL ASPHALT-CONCRETE PAVEMENT

Project No.	Location and Pvt Type	Serration Pattern	Curvature		AADT/1000	Accidents								
			Radius (ft)	Dir.		Before			After			Percent Change		
						Wet	Dry	Tot.	Wet	Dry	Tot.	Wet	Dry	Tot
I-1	07-LA-101 PM7 8-8 8 AC	No serration (control)	Var	Reversing	123	36	59	95	41	75	116	+14	+27	+22
I	07-LA-101 PM8 8-9 3 AC	¼ in. x ¼ in. on 1 in centers-rectangular grooves	2050 2052	Reversing	134	139	55	194	6	35	41	-96	-36	-79

within the curve area. Such action could cause loss of control. The longitudinal grooves by acting as tracks could resist lateral movements and add stability to the vehicle. In the case of the wet pavement condition we may, therefore, assume that longitudinal grooving in curve areas not only increases the friction factor, but also acts as a stabilizer against lateral instability. It probably also serves as a quick surface drain to minimize any water buildup on the pavement.

Table 3 gives the exposure in million vehicle-miles, accident rates, and other information. Both wet and dry pavement accident rates were calculated relative to the number of wet or dry days. These rates could not be calculated at the I-80 location, project M, since the number of wet days was not available.

All of the accident rates on wet days were much higher than the average state highway rates at both urban and rural locations. Because there are relatively few wet days per year in southern California, the resulting exposure is small. When this is divided into the overall large number of accidents occurring on wet pavement, the result is an unusually high rate. All locations (excepting one) had higher than average total accident rates in the before grooving period. The concrete surfaced urban freeways all had below average (< 1.61) rates in the after period. The two rural locations (both concrete surfaced) still had higher than average total accident rates (>1.00) despite sizable drops in rates after pavement serration.

For comparison purposes the accident rate on a one mile stretch of asphaltic-concrete pavement just south of the serrated project N was compared with the unserrated control section. The results are given in Table 4 and clearly indicate the excellent reduction in wet weather accidents following grooving. In the same period the control section had a gain in wet weather accidents.

It is proposed to continue this accident analysis, and periodical skid-resistance surveys to determine possible increase in accidents as the friction values change during service life.

COST OF GROOVING

On seven jobs in District 07 the cost of grooving was in the range of \$0.07 to \$0.09 per sq ft. In some other districts the cost is somewhat higher. The best estimate is approximately \$0.10 per sq ft.

SUMMARY

In summary, it appears that pavement grooving performed in a direction parallel to the centerline will definitely reduce the wet weather accident rate in low friction value areas of PCC pavements. Excellent reduction of wet weather accidents occurred after grooving of an old asphalt-concrete pavement. However, this pavement was very hard and brittle, and we do not recommend grooving of normal asphalt-concrete pavement, since kneading by traffic may rapidly close the grooves. It seems preferable to apply a screening seal coat, slurry seal coat or dense or open-graded blanket.

The friction value is raised following grooving. The rate of change in friction value by wear and polish of the grooved area appears to depend on the characteristics of the original concrete pavement, since two pavements with heavy truck traffic showed little change in friction values after a number of years of service. On the other hand, some pavements show quite rapid drops after only 17 months of traffic. Further tests are required.

Motorcycle and light car tests clearly indicate that ¼- × ¼-in. grooves will create problems in vehicle control. It is recommended that cuts no greater than ⅛- × ⅛-in. be used if vertical grooves are cut in the pavement. However, ⅛-in. deep × ¼-in. wide V-grooves do not appear to create any problems. Further studies are required before any specific spacing may be recommended. However, since approximately equal accident reductions were noted for ½- and ¾-in. spacing, it is recommended that ⅛ × ⅛ in. on ¾-in. centers be used. It is highly desirable that further areas be grooved with a series of patterns as was done on the Ventura project in order to determine effectiveness in raising the original coefficient of friction, and resistance to wear and polish under equivalent concrete and traffic conditions.

ACKNOWLEDGMENT

This paper is based on data collected during a research project financed by the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads. The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

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Appendix

State of California
Department of Public Works
Division of Highways

MATERIALS AND RESEARCH DEPARTMENT

Test Method No. Calif. 342-C
October 3, 1966

METHOD OF TEST FOR PAVEMENT SURFACE SKID RESISTANCE

Scope

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

Procedure

A. Apparatus

1 Skid test unit

a Reference is made to Figures I through III in connection with the following description of the construction of the test unit A 4 80/4 00 x 8, 2-ply tire with (25 ± 2 psi) air pressure (A), manufactured with a smooth surface, together with rim, axle and driving pulley is mounted on a carriage (B) The tire is brought to desired speed by motor (H) The carriage moves on two parallel guides (C), and the friction is reduced to a low uniform value by allowing three roller bearings fitted at 120° points to bear against the guide rod at each corner of the carriage The bearing assembly may be noted on Figure III (D) The two guide rods (C) are rigidly connected to the end frame bars (E) The front end of this guide bar frame assembly is firmly fastened to a restraining anchor The bumper hitch provides for swinging the skid tester to the right or left after positioning the vehicle The rear end of the frame assembly is raised by a special adjustable device (F), Figure II, so as to hold the tire ¼-inch off the surface to be tested This device is so constructed that the tire may be dropped instantaneously to the test surface by tripping the release arm (G), Figure II Tachometer (K) indicates the speed of the tire

2 Hitch for fastening unit to vehicle

3 Special level to determine grade of pavement

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

B. Materials

1 Glycerine
2 Water
3 2-inch paint brush
4 Thickness gauge ¼-inch (a piece of ¼-inch plywood 2′ x 1″ is satisfactory)

C Test Procedure

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

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

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

c The grade is indicated on the calibrated sliding bar

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

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

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

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

6 Set sliding gauge indicator (P) against carriage end

7 Depress starting switch (J) and bring the speed to approximately 55 mi/hr

8 Release starting switch

9 The instant the tachometer shows 50 mi/hr trip arm (G) dropping tire to pavement

10 Read gauge (N) and record

11 Release rebound shock absorber

12 Move to next section and repeat

13 In any one test location, test at 25′ intervals in a longitudinal direction over a 100′ section of pavement

D. Precautions

1 The rear support rod (O), Fig II, must be cleaned by washing frequently with water and a detergent to prevent sticking

2 Sliding gauge indicator (P) must be kept clean so that it will slide very freely

3 On slick pavements glycerine remaining on the pavement should be flushed off with water to prevent possible traffic accidents

E. Field Construction Testing of Portland Cement Concrete Pavement

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

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

2 The determination of test locations, as outlined below, *shall apply only to that portion of the pavement which has well formed striations All areas that appear smooth, or those that have been ground shall be excluded* (See E-3 for procedure to follow for smooth pavements).

a Select a minimum of three test locations for each day's pour and check a minimum of three pour days per contract

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

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

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

Random Numbers Longitudinal	Lateral
0 6	6
0 9	9
0 2	2
0 7	7
0 5	5
0 1	11
0 4	4
0 8	8
0 3	3

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

	Distance from Start of Pour	*Distance from Right Edge of Pour Looking up Station*
Location A	0 6 × 4,000′ = 2,400′	6 × 2 = 12′
Location B	0 9 × 4 000′ = 3,600′	9 × 2 = 18′
Location C	0 2 × 4,000′ = 800′	2 × 2 = 4′

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

At each test location obtain the first reading at the specified random location (using the method described under C-Test Procedure) Obtain the next four readings at 25′ intervals beyond the first reading Obtain all readings at sites parallel to the centerline of the lane After correction for grade as shown in F, average the five readings Record this average as the friction value for the specific test location

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

a Check a minimum of three ground area locations and all smooth appearing surfaces on each contract

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

c If the area is greater than 100′ in length, select sufficient test locations to insure that the area is above the minimum requirement If the average value of all locations is below the required minimum then perform additional tests until the area is localized for remedial action

F Calculations

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

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

Station	*Measured Coefficient of Friction*	*Corrected Coefficient of Friction**
1+00	0 33	0 38
1+25	0 34	0.39
1+50	0 34	0.39
1+75	0 33	0 38
2+00	0 33	0 38
Final Average for Test Site		0 38

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

G. Reporting of Results

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

REFERENCE
A California Method

End of Text on Calif 342-C

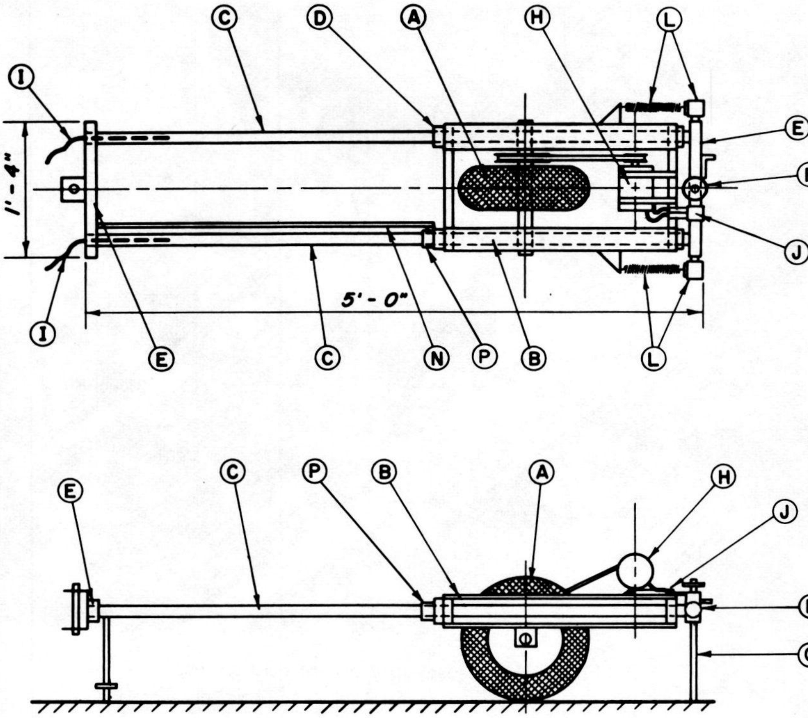

Figure I. Diagram of skid tester.

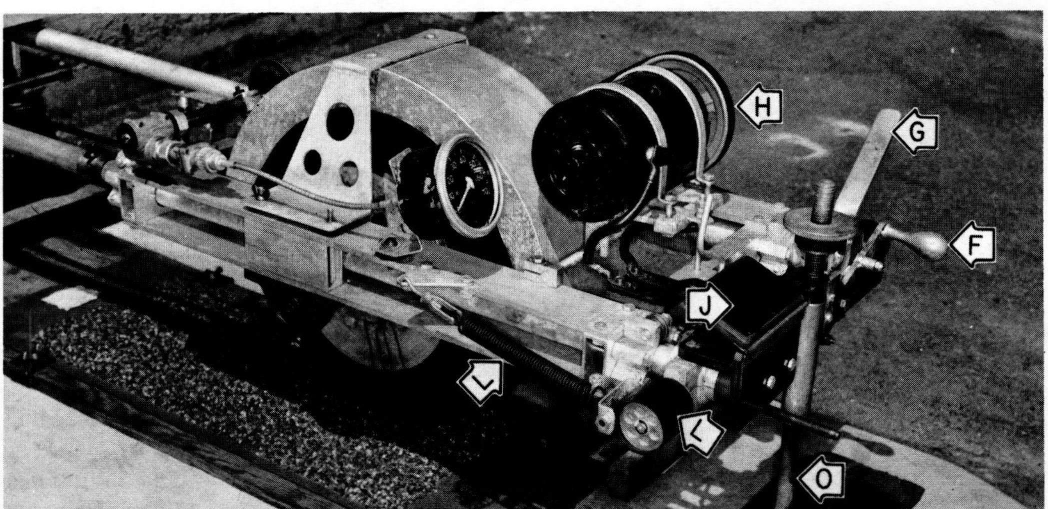

Figure II. Close-up views of skid tester.

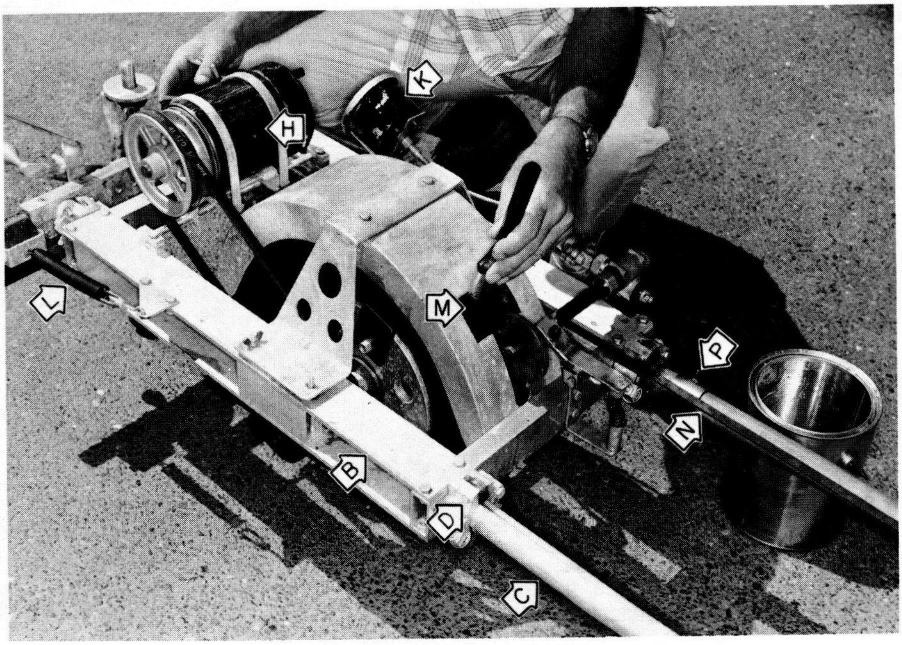

Figure III. Close-up views of skid tester.

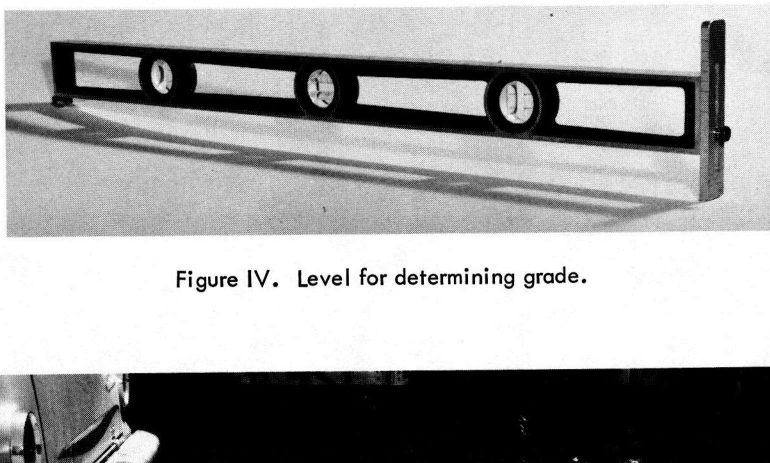

Figure IV. Level for determining grade.

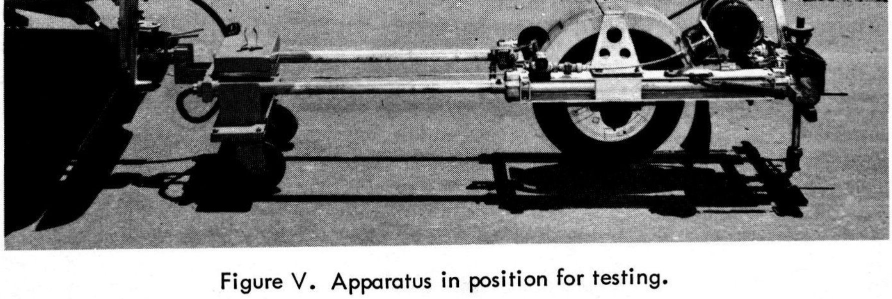

Figure V. Apparatus in position for testing.

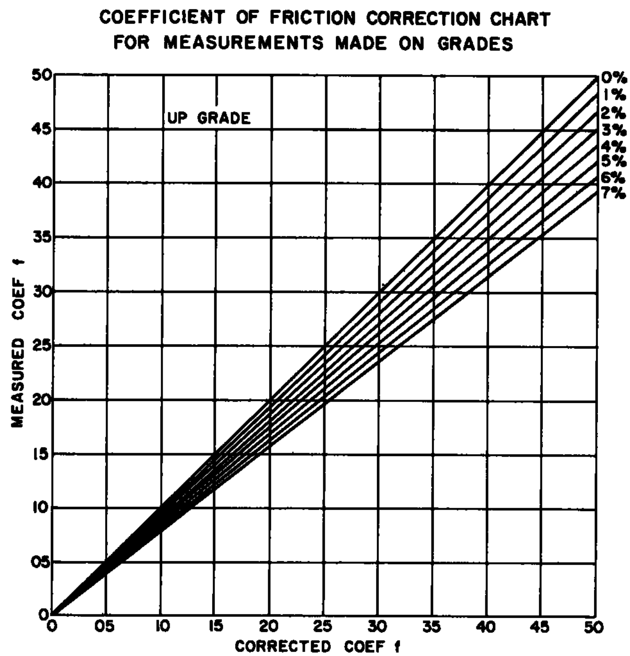

Figure VI. Coefficient of friction correction chart for measurements made on grades.

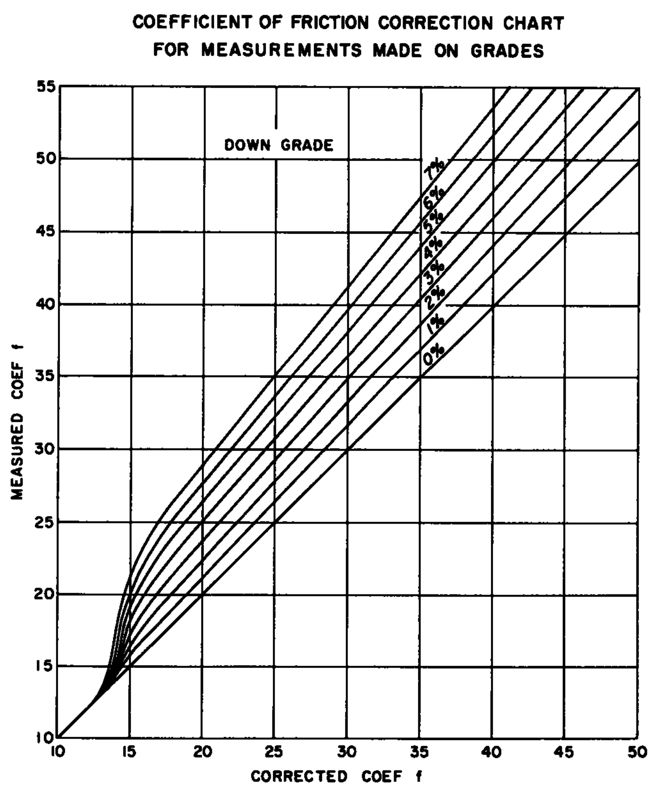

Figure VII. Coefficient of friction correction chart for measurements made on grades.

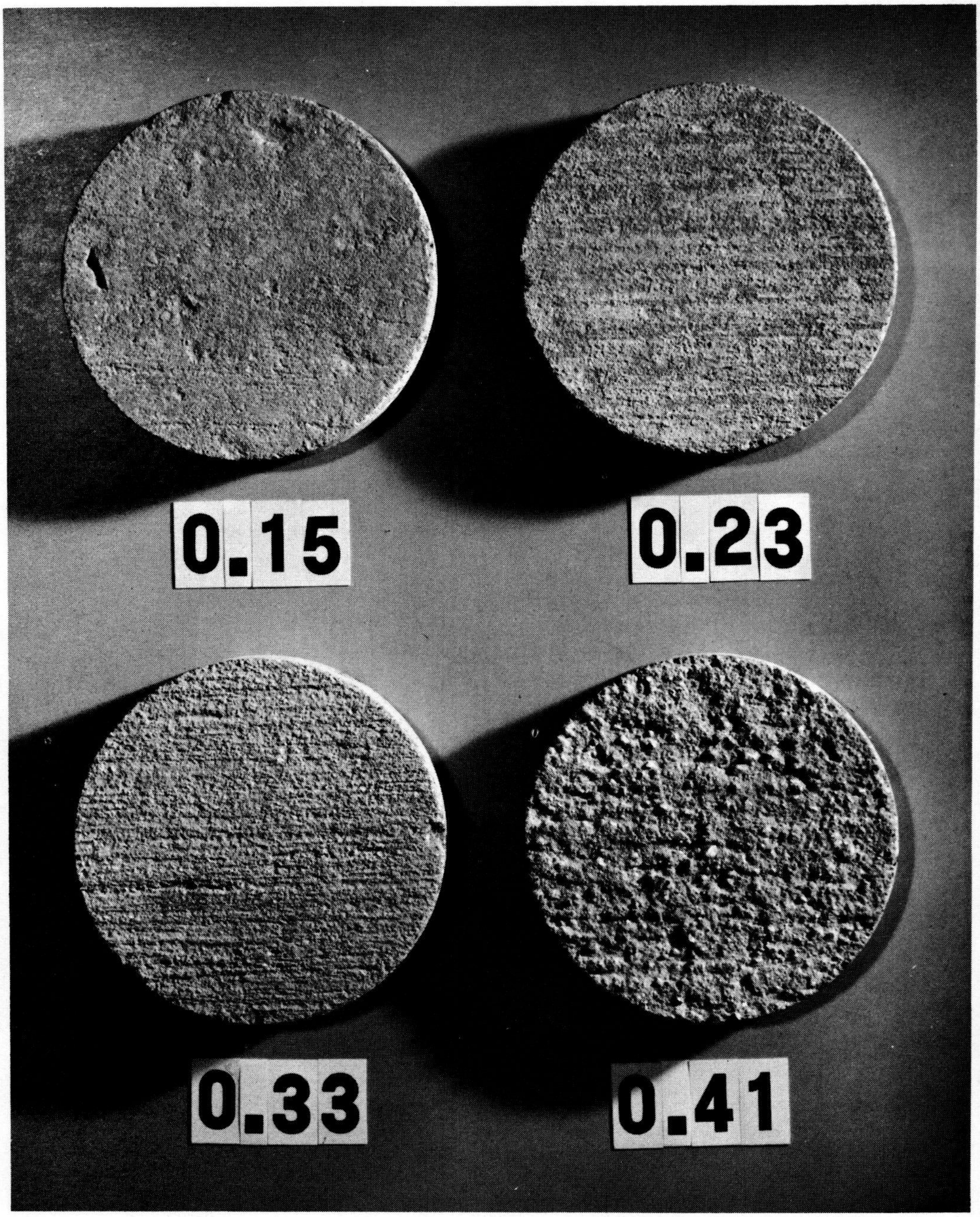

Figure VIII. Photos of surface textures.

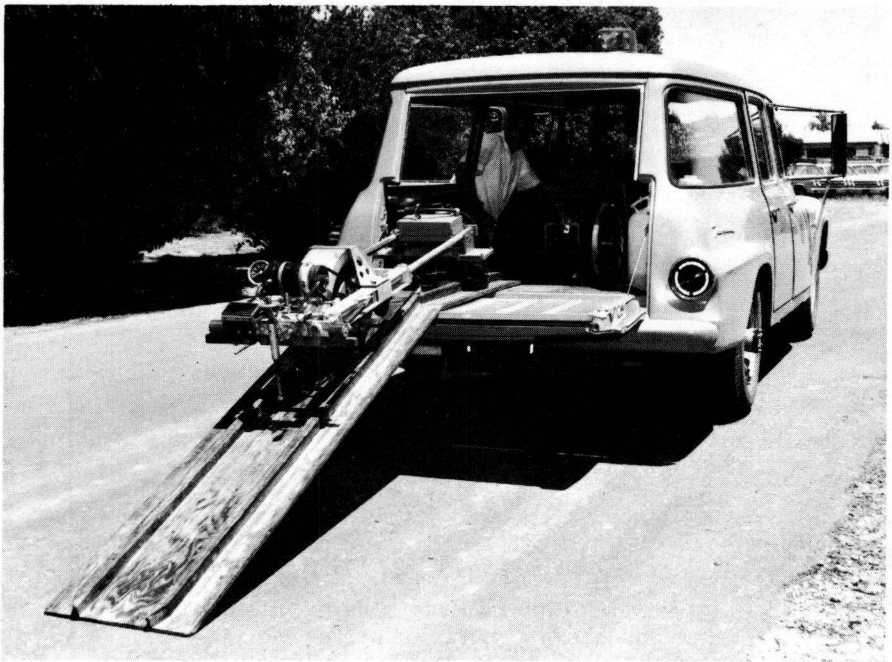

Figure IX. Apparatus being placed in vehicle; note cable and winch for moving skid tester.

Figure X. Apparatus in position for transportation.

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Part II
Asphalt Pavement Cracking

Pavement cracking has concerned pavement engineers and others since the first highway pavement was laid. Generally two types of cracking occur and can be classified as load associated or nonload associated cracking. Each of these types are discussed in four formal papers presented at the 1968 Summer Meeting of the Highway Research Board.

Four informal conference presentations, nonprogrammed presentations, and open floor discussions concerning the papers and the conference session were not recorded and are not included in this report.

Wheel Load Equivalency Based on Flexural Fatigue of Asphaltic Concrete

R. A. JIMENEZ, Arizona Transportation and Traffic Institute, University of Arizona

Wheel load equivalencies have been used in the design of pavements to account for the varieties of load magnitude and configuration. This presentation is concerned with an approach for estimating wheel load equivalencies based on a particular flexure fatigue characteristic of asphaltic concrete. The equivalencies are referred to as destructive ratios, DRs, and are related to the number of repetitions of a particular stress that an asphaltic-concrete surface course can endure in a specified pavement structure. The approach for determining the destructive ratios is described for single-wheel applications of variable loads and tire pressures on pavement structures of different strengths. Comparisons of the calculated destructive ratios were made with wheel load equivalencies established by the AASHO Road Test engineers. These comparisons showed that there was not a consistent relationship between the two equivalencies for the four pavements chosen. However, note is made that in one case results of loads are considered for the surface course only, whereas for the AASHO condition the total pavement structure was involved.

•TRAFFIC on pavements is made up of a variety of wheel loads and gear arrangements. As a consequence, in pavement design, different means (1-6) have been used for obtaining "design wheel loads" or for determining the accumulative damaging effects of different vehicles using the road. It is not the purpose of this report to discuss the various ways of treating mixed traffic for determining wheel load equivalencies but rather to present the results of an approach toward relating the effects of different wheel loads on the fatigue life of asphaltic-concrete surface courses.

FATIGUE OF ASPHALTIC CONCRETE

In recent years there has been great interest in searching for and defining the flexure fatigue characteristics of asphaltic concrete (7-13). A review of the literature on fatigue of asphaltic concrete indicates that the relation between bending tensile stress (S) and number of stress repetitions to cause failure (N) may be expressed as follows:

$$S = I_0 N^{-b} \qquad (1)$$

where I_0 and b are constants. Jimenez (13) reports on fatigue studies of different asphaltic-concrete mixtures and suggests that the value for the exponent, b, in Eq. 1 may vary slightly from -0.2. However, the coefficient, I_0, may vary over a wide range depending on the stiffness or static tensile strength of the mixture. If the slopes of all S-N curves are considered to be the same, then the relative effect on fatigue life of any stress as compared to that of a reference stress, would be constant for asphaltic concretes of different static tensile strengths or I_0's. In this report measure of the relative destructive effect of a stress will be termed destructive ratio, DR, which is the number of load repetitions to cause failure N_1, of the reference stress, S_1, divided by

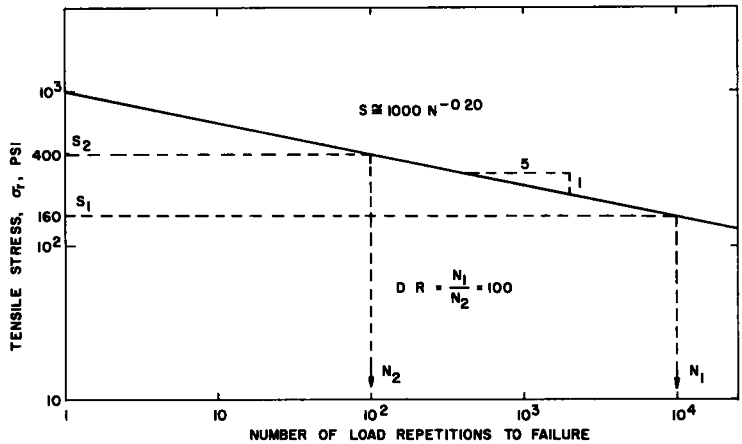

Figure 1. Relationship between tensile stress and number of repetitions to failure.

the number of load repetitions corresponding to failure at the other stress. As an example, consider the following:

From Eq. 1:

$$S = I_0 N^{-b}$$

and

$$N_1 = (S_1)^{-1/b}\ (1/I_0)^{-1/b}$$

$$N_2 = (S_2)^{-1/b}\ (1/I_0)^{-1/b}$$

Then, the DR = $(N_1/N_2) = (S_1/N_2) = (S_1/S_2)^{-1/b}$ which shows that the strength of the material does not influence the value of destructive ratio. Figure 1 shows the above concept where S_1 is the reference stress.

The calculations for stresses given in Table 1 are based on asphaltic concrete having a static bending strength value, I_0, of 1,000 psi.

STRESSES AND PAVEMENT STRUCTURES

Asphalt pavements may be considered as three-layered systems, and the stresses at certain locations can be calculated by use of Jones' tables (14). The stresses of concern in this report are the central radial stresses at the bottom of the surface course.

The structural systems examined for this presentation are shown in Figure 2 and are listed as follows:

1. Surface course of asphaltic concrete
   (a) Modulus E_1 = 50,000, 100,000, and 125,000 psi.
   (b) Thickness h_1 = 1, 2, 4, and 8 in.
2. Base
   (a) Modulus E_2 = 5,000 psi.
   (b) Thickness h_2 = 8 in.
3. Subgrade
   (a) Modulus E_3 = 2,500 psi.

The selections of E_1 were somewhat arbitrary but appear to be reasonable in comparison with such values as previously given (13). The values of K_1 or E_1/E_2 were selected to yield ratios of 10, 20, and 25. This then fixed the value of 5,000 psi for E_2

TABLE 1

STRESSES AND DESTRUCTIVE RATIOS, DR, FOR VARIOUS SINGLE WHEEL LOADS AND PAVEMENTS
(h_2 = 8 in., E_2 = 5,000 psi, and E_3 = 2,500 psi)
$S = 1000\ N^{-0.2}$

Load and Pres. (kips - psi)		h_1 (in.)	K_1 = 10				K_1 = 20				K_1 = 25			
			S_{r1} (psi)	N ($\times10^3$)	DR N_1	DR 820	S_{r1} (psi)	N ($\times10^3$)	DR N_1	DR 820	S_{r1} (psi)	N ($\times10^3$)	DR N_1	DR 820
4	60	1					221	2.4	6.14	0.34				
6	60						174	8.1	1.80	0.10				
8	75						162	11.7	1.24	0.07				
9	75						155	14.6*	1.00	0.06				
10	80						150	17.0	0.86	0.05				
12	90						141	24.0	0.61	0.03				
4	60	2	118	58.5	0.12	0.01	243	1.4	0.14	0.58	300	0.5	0.15	1.69
6	60		140	24.5	0.29	0.03	288	0.6	0.33	1.36	353	0 2	0.35	3.81
8	75		166	10.8	0.66	0.08	333	0.3	0.69	2.83	406	0.1	0.73	8.05
9	75		178	7.1*	1.00	0.11	357	0.2*	1.00	4.10	432	0.075*	1.00	10.91
10	80		190	5.1	1.40	0.16	378	0.1	1.33	5.46	459	0.05	1.44	15.73
12	90		216	2.6	2.73	0.31	423	0.08	2.41	9 88	511	0.03	2.38	26.21
4	60	4	106	102.4	0 10	0.01	160	12.2		0.07	179	7.0	0.05	0.12
6	60		130	36.1	0.29	0.02	205	3.5		0.24	233	1.8	0.21	0.46
8	75		154	15.1	0.69	0.05	250	1.3		0.65	287	0.6	0.62	1.34
9	75		166	10.4*	1.00	0.08	271	0.82*		1.00	314	0.39*	1.00	2.10
10	80		178	7.1	1.45	0.11	293	0.5		1.51	341	0.3	1.49	3.21
12	90		203	3.7	2.81	0.22	337	0.3		3 06	395	0.1	3 19	6.89
4	60	8					56	1860.0	0.02					
6	60						84	316.0	0 14					
8	75						112	78.2	0.55	0.01				
9	75						127	42.8*	1.00	0.02				
10	80						141	24.0	1.79	0 03				
12	90						169	9.3	4.60	0.09				

*Reference repetitions (N_1)

which will be considered to represent an unbound base course material. According to Peattie (15), "... for unbound granular materials the effective value of E_2/E_3 invariably lies between 2 and 5." The selection of E_2/E_3 = 2, thus sets our value for E_3 equal to 2,500 psi. It is apparent that the absolute values of E's are not a primary factor for stress computations; but the modular ratios are.

The use of Jones' tables for stress calculations required that circular and uniformly loaded contact areas be assumed. With respect to the larger wheel loads that would normally be carried on duals, it was assumed that the area of contact with the pavement surface was circular and equal to the load divided by the tire inflation pressure. Additionally, it was considered reasonable to increase the tire inflation pressure as the wheel load increased. A review of truck tire ratings given by the Tire and Rim Association (16) suggested the grouping for tire pressures and loads given in Table 2.

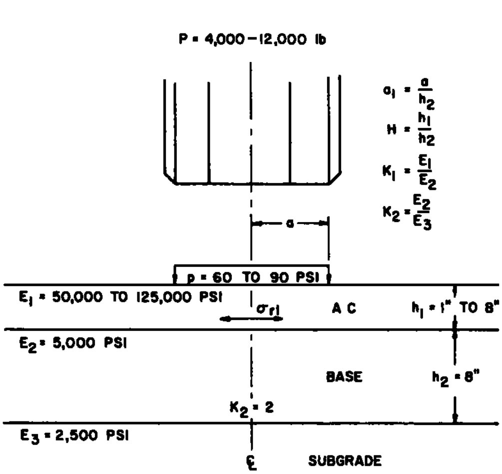

Figure 2. Variables and symbols for three-layered systems.

Typical calculated tensile stresses resulting from the selected wheel loads are shown in Figure 3. The data points do not lie on a straight line; however, for practical purposes and especially in consideration of the assumed contact areas, a linear relationship between the tensile stress and single wheel load was accepted as shown by the solid lines. As expected, the higher the K_1 value, the higher was the

TABLE 2

TIRE PRESSURE AND LOAD GROUPING

Total Load (lb)	Inflation Pressure (psi)
4,000	60
6,000	60
8,000	75
9,000	75
10,000	80
12,000	90

stress for other conditions fixed. Figure 3 also shows that the rate of stress increase with increasing load is greatest for the highest K_1 plot.

In Figure 4, effects of surface thickness on the relationship between tensile stress and load are presented. The relative position of the h = 1-in. curve is interesting. From the low stresses associated with this curve it would seem that the maximum curvature of the deflected layer does not occur directly below the center of the loaded area and/or that a greater portion of the load is transmitted to the base course.

DESTRUCTIVE RATIO

Stresses and destructive ratios for the different pavement structures and also reference stresses are listed in Table 1. Typical relationships between destructive ratio and wheel load are shown in the semi-log plot of Figure 5. The reference condition is a 9,000-lb wheel load on a pavement with a surface course thickness of 4 in. and a K_1 value of 20. The relative positions of these curves indicate the changes in the damaging effects of different wheel loads as the base loses stiffness in comparison to the surface course (E_1/E_2 increases), such as may occur during a spring thaw or when the asphaltic concrete increases in stiffness due to temperature changes or aging of the asphalt.

In order to check this approach for establishing wheel load equivalencies, comparisons were made with comparable values determined at the AASHO Test Road (4). Such a comparison is shown in Figure 6 for a pavement with a common structural number of 3.0. The SN of 3.0 was obtained using a_1 and a_2 values of 0.44 and 0.14, respectively. The plot of fatigue equivalency vs. AASHO equivalency shows almost a perfect agreement. It should be remembered that equivalencies from the AASHO data were related to the total pavement structure and not to the surface course only as is the case for the fatigue equivalencies.

Further comparisons of equivalencies with AASHO values are shown in Figure 7. There is not a consistent relationship between the AASHO and fatigue equivalencies for

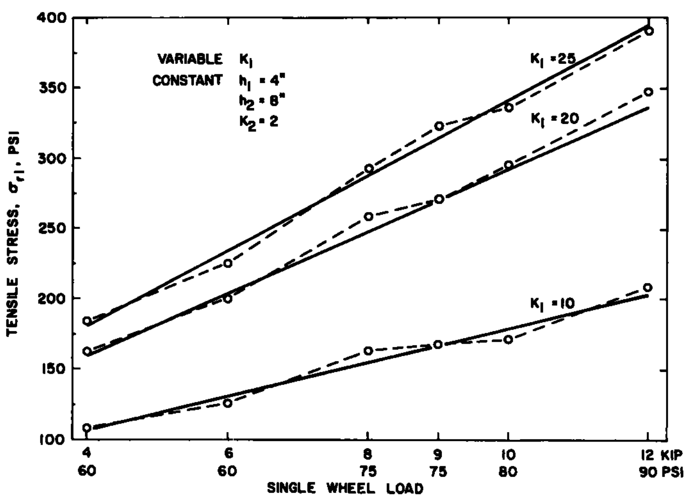

Figure 3. Relationship between tensile stress and wheel load.

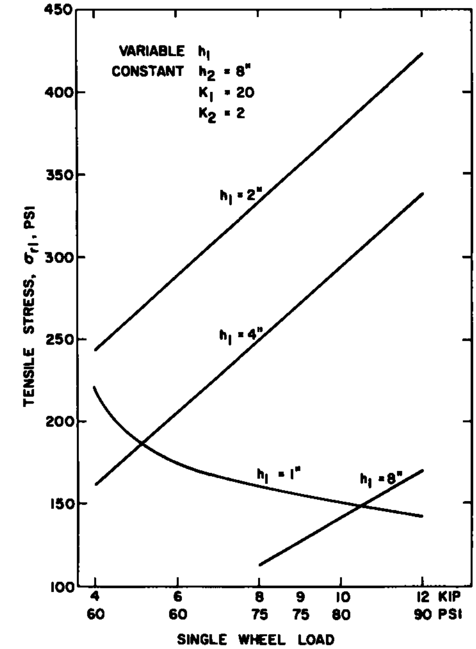

Figure 4. Relationship between load and tensile stress at 1st interface.

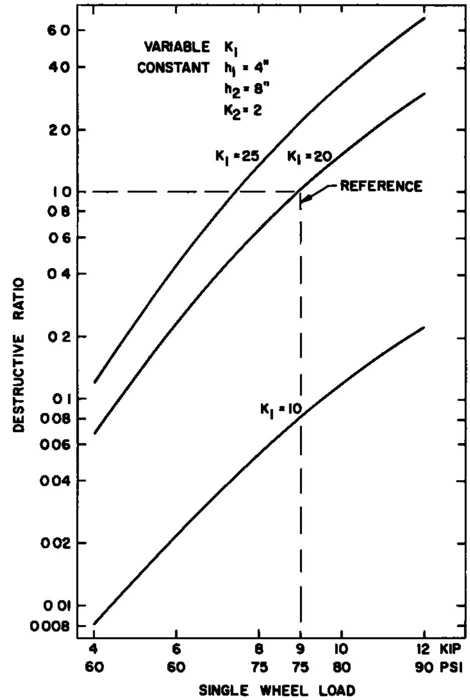

Figure 5. Relationship between wheel load and destructive ratio.

the pavements with surface course of different thicknesses. From Figure 7 and considering the assumed conditions of pavements and wheel loads, the following conclusions are reached:

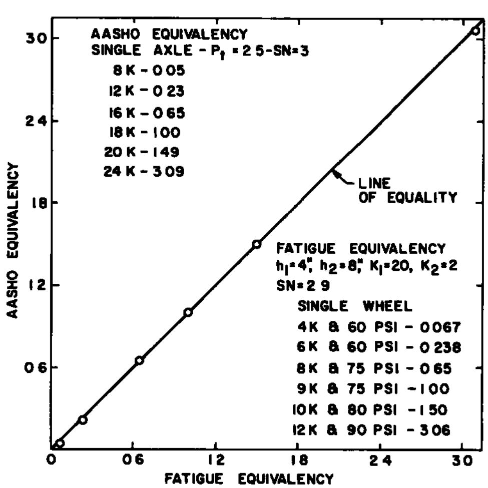

Figure 6. Relationship between AASHO and fatigue wheel load equivalencies.

1. For the pavement with the 1-in. thick surface course, the larger wheel loads are less destructive with respect to flexure fatigue of the surface; however, these loads would have more damaging effects to the subsoils.

2. For the pavement with the 2-in. surface course, single wheel loads up to 9,000 lb are more damaging to the surface than to its foundation and then as the load increases it contributes more to subsoil failure.

3. For the pavement with the 4-in. surface course, the wheel loads are equally damaging to the surface course and to the subsoils.

4. For the pavement with the 8-in. surface course, single wheel loads up to 9,000 lb are slightly less damaging to the surface course than to its foundation and then as the load increases it contributes more to flexure fatigue.

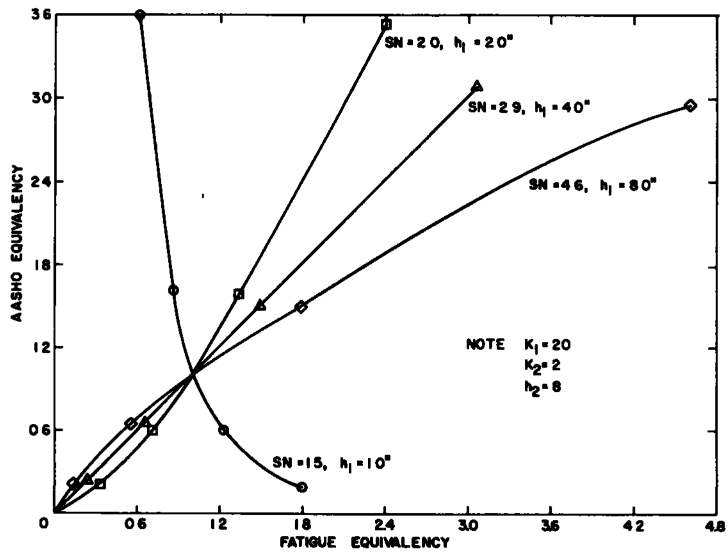

Figure 7. Relationship between AASHO and fatigue wheel load equivalencies.

SUMMARY

This discussion has been quite limited and perhaps too much liberty has been taken for the conditions assumed. Nevertheless, the approach presented for determining wheel load equivalencies appears reasonable and with the comparisons of these values with the equivalencies established by AASHO serve to warrant the following:

1. Flexural fatigue characteristics of asphaltic concrete are a factor to consider in determining wheel load equivalencies.
2. The changes that occur in relative stiffness (E_1/E_2) between a surface course and a base will affect the damaging effects of a wheel load.
3. For thin surface courses (< 1.5 in.), the smaller wheel loads can be more destructive than the larger ones.
4. The destructiveness of different wheel loads may be greater with respect to flexure fatigue strength of the asphaltic concrete than with respect ot the strength of foundation soils.

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Thermal Fracture Phenomena in Bituminous Surfaces

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This paper explores the phenomenon of thermally induced cracking in bituminous surfaces as a problem primarily associated with design. Low-temperature effects on pavement performance are examined and the need for modifying current mixture design procedures is discussed. Since any such design requirements would be concerned with fracture susceptibility, an approximate procedure for calculating thermally induced stresses is presented which recognizes the probable stiffness and temperature gradients that occur in field service.

A consideration of fundamental mechanisms involved is essential to adequately fulfilling design objectives; consequently, an hypothesis has been advanced to the effect that thermally induced cracking occurs in two main phases. These consist essentially of limited-depth crack initiation, and subsequent full-depth propagation with rising air temperatures. The pertinent mechanics are explained and the hypothesis appears to be compatible with observed phenomena.

Several practical implications of the hypothesis are discussed and these relate primarily to the development of mix designs to mitigate or reduce cracking and to the possible use of "good" and "poor" asphalts in the same pavement structure. It is suggested that more efficient and economical uses of available materials may be possible in designing for the low-temperature cracking problem.

•THE pavement design problem has received much attention during the past two decades and a considerable amount of information is available. Yet, we are in many ways little closer to understanding what this problem really is, either in a general sense or a specific case. Broadly, we recognize that the principal objective of a highway pavement is to provide for the safe, efficient and economical passage of road vehicles under all climatic conditions. However, we are in most cases unable to satisfy this objective as economically and efficiently as we desire, largely due to a state of imperfect technology. This involves an incomplete understanding of both the fundamental behavior and control of pavement component materials, or of their combined state in a pavement structure, and their contributions to the performance of the system as a whole under a variety of traffic, climatic and age conditions.

Hutchinson and Haas (1) have considered this situation in setting forth a systems analysis of the total highway pavement design process. Their analysis and recommendations are based on the premise that a completely rationalized process can only be developed through the use of techniques that permit the serviceability-age histories of alternate designs to be predicted along with the total cost streams necessary to produce these serviceability profiles. Additionally, their framework recognizes the need for a systematic identification of all performance variables and the placing of any one component's performance within the context of the system's performance as a whole. This requires an attendant advancement of the state of technology to the point where a component material, such as the bituminous mixture, can truly be "designed"; i.e., for any proposed pavement design, a bituminous material input for a specified performance requirement can be produced with predictable economic implications.

The intention of this paper is to explore the phenomenon of thermally induced fracture in bituminous surfaces with respect to:

1. The variables in low-temperature cracking,
2. Mixture design considerations for low-temperature conditions,
3. Calculating thermally induced stresses,
4. Defining the mechanisms of fracture initiation and propagation.

The major limitation of the paper is that the mechanisms are essentially postulated, although logically supported, and require experimental verification. Nevertheless, since the hypothesis may have some significant practical implications, it is considered useful to present it at this time in the hope that it will stimulate the necessary experimentation.

LOW-TEMPERATURE EFFECTS ON PAVEMENT PERFORMANCE

Advancing the state of technology to the point where a pavement component material can truly be designed implies a knowledge of the fundamental mechanisms that control in-service behavior. Only with this type of information can the design techniques mitigate any unsatisfactory effects of such behavior.

Low-temperature cracking is one of these unsatisfactory aspects of the overall behavior of flexible pavements. At the time of occurrence, it has relatively little effect on pavement riding quality. However, its implications, or "secondary deterioration" effects, such as bumps, dips, spalling, and potholes, can be highly detrimental to the performance and useful life of the pavement structure. The severity of this has been demonstrated by recent studies in Alberta (2), Saskatchewan (3, 4), Manitoba (5), Ontario (6, 7), Wyoming (8) and Connecticut (9). A result of these studies is that the primary mechanism of low-temperature cracking is receiving increased attention; some of this attention involves work on fundamental fracture phenomena (10). However, before we examine such fracture phenomena, several significant aspects must be appreciated; they include the following:

1. There can be a number of causes of fracture in bituminous mixtures; low-temperature shrinkage is only one of these.
2. The effects of asphalt aging on low-temperature physical properties of bituminous mixtures are very imperfectly understood.
3. Present practice in designing bituminous mixtures for physical behavior considers only higher temperatures (i.e., usually 140 F).

Low-Temperature Cracking

Pavements can deteriorate due to a variety of cracking types or other degradation under traffic and environmental factors. Low-temperature cracking itself of bituminous surfaces can also occur due to several causes that include:

1. Thermal stresses in the bituminous surface exceeding the tensile strength, without considering traffic loads;
2. Freezing shrinkage and cracking of the subgrade and propagation through the bituminous surface; and
3. Thermally induced stresses in the bituminous surface, coupled with traffic imposed stresses (i.e., cold pavement but unfrozen base and subgrade with "normal" deflection bowl under traffic), exceeding tensile strength.

Effects of Asphalt Aging

A considerable amount of effort has gone into studying in-service asphalt aging characteristics. Most of this work, however, has examined only the "average" effect throughout the depth of the pavement by extracting asphalt from core samples. Very little work has been done on "aging gradients" although it has been suspected for some years that the asphalt at the top of the layer can be significantly harder than at some depth, depending on how long the pavement has been in service. Kari and Santucci (11) have shown this phenomenon to exist on the basis of viscosity measurements but their work was primarily related to air void variations with depth on relatively new pavements. Recently, The Asphalt Institute has conducted some tests on the Virginia Test Road (12), and found, on the basis of fundamental viscosity at 140 F, that the extracted asphalt at the surface was up to ten times harder than at the bottom of the layer. As far as low-temperature behavior is concerned, the question of how such asphalt aging may affect cracking susceptibility has not yet apparently been investigated. It seems reasonable to assume though that it could be a significant factor in creating a stiffness gradient at the lower temperatures.

Low-Temperature Mix Design

Current methods of designing bituminous paving mixtures look for an end product that has a certain minimum stability at the maximum expected service temperature (usually 140 F), a deformation between certain limits at this temperature and certain air void and voids in mineral aggregate ranges. Additionally, we place various physical and chemical specification restrictions on the binder and aggregate components themselves. All these are supposed to bear some relationship to the in-service performance of the asphalt-aggregate system; yet, these criteria, most of which are high temperature oriented, are in reality little related to the range of environmental and loading conditions the material will be subjected to in its service life. One major reason has likely been the somewhat implicit assumption that high-temperature stability, deformation, skid resistance and durability were by far the most important considerations and that rapidly increasing stability with decreasing temperature negated the requirements for any design criteria in this area. Yet, the need for low-temperature design modifications in some regions was recognized by Rader (13) over 30 years ago when he stated, "Quantitative data obtained from low temperature tests should aid engineers in solving

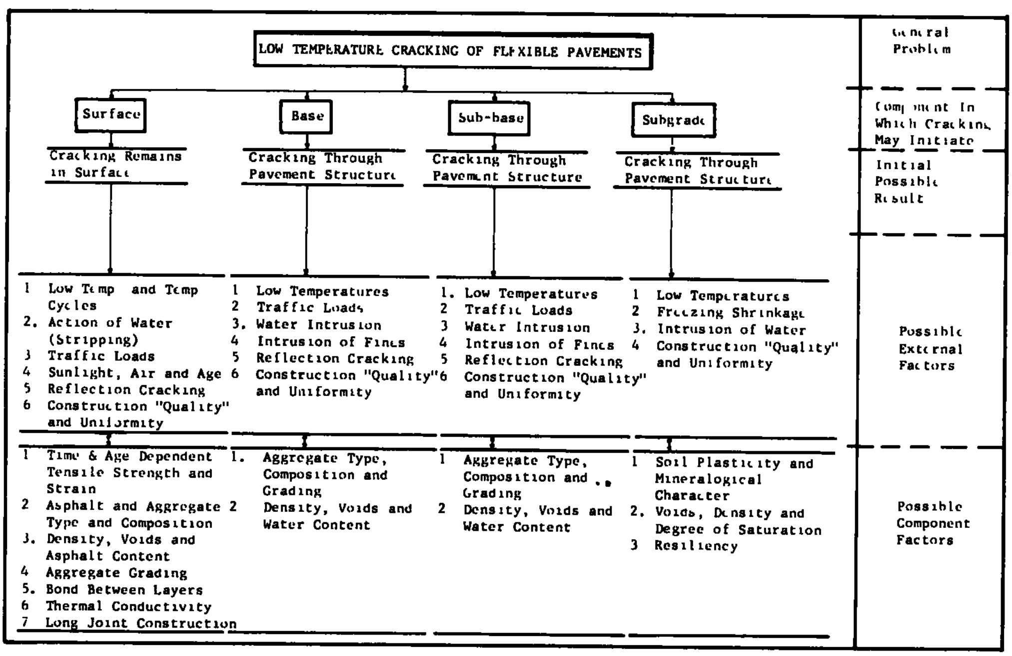


Figure 1. Factors of possible significance in low-temperature cracking of flexible pavements.

problems relating to the control of cracking." The appreciation of this need seems to have lain relatively dormant though until the markedly increased attention given to low-temperature cracking during the past five years. Consequently, except for some recent work in Alberta (14), design supplements or modifications for bituminous concretes in cold weather applications have not yet been developed.

Figure 1 summarizes the variety of factors that can be significant to low-temperature cracking. It illustrates that the variables can be external or related to the character of the component. The basic problem with respect to the bituminous materials is to identify and evaluate the component factors, in the light of the external factors, and to exercise a degree of control that maximizes the economic benefits accruing from reduced cracking.

THERMAL STRESSES IN BITUMINOUS SURFACES

Previous Efforts

Any comprehensive examination of thermal fracture phenomena in bituminous surfaces must necessarily consider the calculation of thermally induced stresses and a comparison with time and temperature dependent tensile strength of the mixture. Several attempts have been made to calculate such stresses and they include those by Monismith et al (15), Lamb et al (16), and Hills and Brien (17).

The first of these (15) utilized a stress equation developed by Humphreys and Martin (18) for an infinite slab composed of a linear viscoelastic material, subjected to a time-dependent temperature field. The equation provides for the calculation of the horizontal tensile stress field, as a function of depth, time and temperature. Temperature distribution was determined from a solution of the heat conduction equation, rather than from experimental field data. However, temperature distribution is merely an input to the stress equation; consequently, the method of determining it is of no concern insofar as solution of the stress equation is concerned. Relaxation moduli for inclusion in the equation were determined from the conversion of creep compliances from uniaxial creep test data.

The second stress calculation method (16) is much more simple in an analytical sense and considers the pavement surface as an elastic, infinitely long beam of finite width. For this case, where restraint is considered in two directions and no temperature gradient exists through the depth of the layer, the unit tensile stress in the longitudinal direction is given by:

$$\sigma_x(T) = E\alpha(\Delta T) + \mu f L$$

where

- E = Young's modulus;
- α = average coefficient of thermal contraction, over the temperature drop, ΔT;
- μ = Poisson's ratio for the material;
- f = coefficient of friction between the bituminous layer and the base; and
- L = one half the width of the pavement section.

Now, if E is replaced by a stiffness modulus, S, which varies with temperature and time of loading, a more accurate but still approximate value of longitudinal stress may be obtained. Lamb and his co-workers evaluated this modulus for the midpoint of their most extreme daily temperature drop range in the winter and obtained tensile stresses as high as 110 psi. The contribution of lateral restraint to this stress was found to be relatively minor.

The third method (17) has extended the concept of stiffness modulus over the entire temperature range and utilizes experimentally determined values of the modulus at the midpoint of small discrete temperature intervals. The equation for longitudinal stress, for the same infinitely long beam (neglecting lateral restraint) then becomes

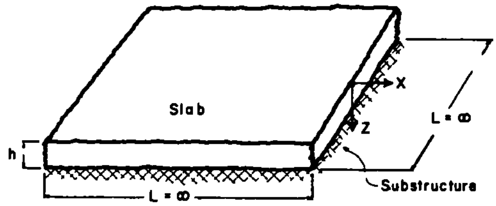


Figure 2. Infinite slab of viscoelastic material bonded to a rigid substructure.

$$\sigma_x(T) = \alpha \sum_{T = T_0}^{T = T_f} [S(\Delta T)]$$

where

S = stiffness modulus, determined experimentally, at midpoint of each ΔT; and
ΔT = temperature interval, of which a finite number are taken between $T = T_0$ and $T = T_f$.

Hills and Brien have used this equation to calculate thermally induced stresses in a beam of pure asphalt where the stiffness modulus was obtained at the midpoint of small temperature intervals, using a loading time corresponding to the cooling time for the temperature interval. They contend that any error involved in this use of corresponding times is likely to be small. The stress values obtained were then compared to tensile strengths and probable fracture temperatures determined. The actual temperatures for fracture, by direct measurement, were found to be in relatively good agreement, for a limited range of experimental work and no temperature gradients.

A Rigorous Approach to Determining Stresses

A rigorous analysis of thermally induced stresses can consider the material as viscoelastic in nature and this has been done by Humphreys and Martin (18) in considering an infinite flat slab bonded to a rigid substructure layer. This condition is shown in Figure 2 and the general field equations, in the absence of temperature dependence, used in their analysis may be listed as follows:

Strain Tensor:

$$\epsilon_{ij}(x_k, t) = \frac{1}{2}\left[\left(\frac{\partial}{\partial x_j}\right) u_i(x_k, t) + \left(\frac{\partial}{\partial x_i}\right) u_j(x_k, t)\right]$$

Equilibrium Condition:

$$\left(\frac{\partial}{\partial x_j}\right) \sigma_{ij}(x_k, t) = 0$$

Stress Strain:

$$s_{ij}(x_k, t) = \int_{-\infty}^{t} G_1(t - \tau)\left(\frac{\partial}{\partial \tau}\right) e_{ij}(x_k, \tau)\, d\tau$$

$$\sigma(x_k, t) = \int_{-\infty}^{t} G_2(t - \tau)\left(\frac{\partial}{\partial \tau}\right)\left[\epsilon(x_k, \tau) - 3\alpha\theta(x_k, \tau)\right] d\tau$$

where

ϵ_{ij} = strain tensor;
σ_{ij} = stress tensor;
u_i, u_j = displacements;
t = time;
s_{ij} = deviatoric components of the stress tensor;
G_1, G_2 = general relaxation moduli;
τ = a time variable for integration;
e_{ij} = deviatoric components of the strain tensor;

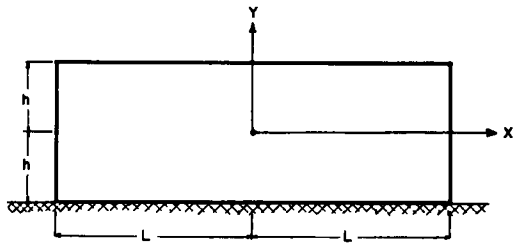


Figure 3. Finite plate of elastic material bonded to a rigid substructure.

σ = hydrostatic stress = σ_{kk};
ϵ = hydrostatic strain = ϵ_{kk};
α = coefficient of thermal expansion or contraction; and
θ = temperature change = $T(z, t) - T_0$.

The principle of time-temperature superposition has been used by Humphreys and Martin in obtaining the general constitutive equations which in turn permit the use of a "reduced time," ξ, in the last two equations. The application of a Laplace transform procedure to these modified equations, and the use of various stress and strain boundary conditions for an infinite slab in the first two equations, leads to the desired stress equation:

$$\sigma_{xx}(z, t) = -3\alpha_0 \int_0^t R\left[\xi(z, t) - \xi(z, \tau)\right]\left(\frac{\partial}{\partial\tau}\right)\theta(z, \tau)d\tau$$

where

$\sigma_{xx}(z, t)$ = horizontal stress in any direction, as a function of depth z, and time, t, for a slab of infinite extent in all horizontal directions (Fig. 2).

This stress equation was used by Monismith et al (15) in their analysis, which includes a definition of terms and an explanation of the numerical solution used. They point out, however, that the computed stresses, which are as high as about 3,300 psi for a particular case, may be somewhat greater than those which actually occur. This results from the assumption of infinite extent in the lateral direction, whereas the pavement may be more realistically simulated by an infinitely long strip. Consequently, although it is beyond the intention of this paper, there seems to be scope for exploring the case of such a strip, with its associated boundary conditions, using the general field equations quoted. The applicability of this type of approach could be checked in the laboratory with specimens of suitable geometry that are restrained in a manner simulating that of the prototype and subjected to a variable temperature field.

The determination of thermal stresses in restrained elastic plates has received attention from a number of authors, including a recent effort by Iyengar and Chandrashekhara (19). Their analysis assumed an isotropic and homogeneous material, uniform temperature change, no temperature variation with thickness, elastic constants independent of temperature, and a state of plane stress. The problem was formulated in terms of Airy's stress function. This involves the determination of a stress function, ϕ, which satisfies the following equation:

$$\frac{\delta^4\phi}{\delta x^4} + 2\frac{\delta^4\phi}{\delta x^2\,\delta y^2} + \frac{\delta^4\phi}{\delta y^4} = 0 \tag{1}$$

where the x and y directions are as shown in Figure 3.

The stress component in the longitudinal direction, σ_x, which would be of prime interest for a pavement, is determined from

$$\sigma_x = \frac{\delta^2\phi}{\delta y^2} \tag{2}$$

and the displacement component in the x direction (for the plane stress case) is determined from

$$u = \left(\frac{1+\nu}{2}\right)\frac{\delta\phi}{\delta x} + \frac{1}{E}\,\frac{\delta\psi}{\delta y} + \alpha$$

where

ν = Poisson's ratio for the material;
E = modulus of elasticity;
α = coefficient of thermal expansion or contraction;
ψ = a displacement function, defined as $\delta^2\psi/\delta x\,\delta y = \nabla^2\phi$ and $\nabla^2\psi = 0$.

Iyengar and Chandrashekhara (19) have considered the following expression for the stress function which satisfies the differential equation (Eq. 1), as well as the boundary conditions of zero longitudinal stress at the ends of the plate and zero normal stress at the free surface of the plate.

$$\begin{aligned}
\phi = & \sum_{n=1,3}^{\infty} \frac{A_n \cos \beta_n x}{\beta_n^2 \sinh \beta_n h}\left[\beta_n y \sinh \beta_n y - \beta_n h \tanh \beta_n h \cosh \beta_n y\right] \\
& + \sum_{n=1,3}^{\infty} \frac{B_n \cos \beta_n x}{\beta_n^2 \cosh \beta_n h}\left[\beta_n y \cosh \beta_n y - \beta_n h \coth \beta_n h \sinh \beta_n y\right] \\
& + \sum_{s=1,3}^{\infty} \frac{C_s \sin \delta_s y}{\delta_s^2 \sinh \delta_s L}\left[\delta_s x \sinh \delta_s x - \delta_s L \tanh \delta_s L \cosh \delta_s x\right] \\
& + \sum_{r=1}^{\infty} \frac{D_r \sin \gamma_r y}{\gamma_r^2 \sinh \gamma_r L}\left[\gamma_r x \sinh \gamma_r x - \gamma_r L \tanh \gamma_r L \cosh \gamma_r x\right]
\end{aligned} \tag{3}$$

where

A_n, B_n, C_s, D_r = Fourier constants;
$\beta_n = \frac{n\pi}{2L}$ where n = 1, 3, 5,...;
$\delta_s = \frac{s\pi}{2L}$ where s = 1, 3, 5,...;
$\gamma_r = \frac{r\pi}{h}$ where r = 1, 3, 5,....

Using boundary conditions for shear stresses and displacements appropriate to two particular cases, Iyengar and Chandrashekhara have shown how, after finite Fourier transformation and simplification, expressions can be determined for the four Fourier constants. Simultaneous solution of these expressions, using a finite number of terms for each series, makes it possible to obtain quite accurate values for the four unknowns. A numerical value for σ_x can then be found by differentiating Eq. 3, according to Eq. 2, and evaluating the result.

The foregoing analysis applies to uniform temperature through the plate; however, arbitrary temperature variation can be handled following the same procedure and using a general solution according to these same writers. Additionally, the solution is for the elastic case where the modulus, E, does not change with time. It may be feasible, however, to use a finite difference method by establishing E experimentally as a time and temperature dependent modulus for each small temperature decrement, similar to the previously mentioned approach of Hills and Brien (17). Then, the use of the more rigorous method of employing a stress function, for the elastic case, can be explored for varying boundary conditions and a finite length of pavement in terms of various L/h ratios (Fig. 3).

An Approximate Method for Determining Stresses

Stresses in an asphalt beam may be approximately determined, as previously pointed out, by the use of the equation for elastic stresses, with a time and temperature dependent stiffness modulus substituted for the modulus of elasticity. This modulus can be determined at the midpoint of small, discrete temperature intervals and the maximum tensile stress found by summating stress increments over the total temperature range. However, it is well known that due to the relatively low thermal conductivity of asphaltic concrete, significant temperature gradients usually exist through the depth of the surface layer. This has been well demonstrated by some recent work of The Asphalt Institute (20) and the Clarkson Institute of Technology (21), but very few such data exist on temperature variations in extreme cold weather areas. Nevertheless, it seems that more accurate determinations of temperature induced stresses and fracture susceptibility should definitely take the temperature gradient into account, as subsequently demonstrated in this paper. Additionally, the possibility of a "stiffness gradient," due to greater hardening of the asphalt binder at the surface (and exclusive of that created by the temperature gradient), may be taken into account as a further refinement.

The following listing of several possible "cases" considers these factors in extending the stiffness modulus concept to the determination of stresses at the top and the bottom of the bituminous layer. Intermediate stresses could be determined if the temperature and stiffness gradients were known through the depth of the pavement layer. Figure 4 shows a schematic representation of these cases for additional clarity. The stress calculations shown make the following simplifying assumptions:

1. The effect of lateral restraint on longitudinal stresses is omitted;
2. A linear temperature gradient prevails for certain initial intermediate or final conditions, which may be an oversimplification in some cases, as indicated by Straub's work (21); and
3. A linear stiffness gradient prevails, for certain conditions, which may or may not be reasonable since no data exist in this regard.

As well, to account for the time rate of temperature change, dT/dt, not being the same at the top and bottom of the layer in some cases, the summations of stress increments are shown over different total temperature intervals (although equal temperature decrements, ΔT, are used in all cases).

Case 1: The beam is subjected to a uniform temperature drop, with no temperature variation with depth, from T_o to T_f with a constant stiffness modulus through the depth at any one temperature. In this case, the maximum thermally induced, longitudinal stress, at any depth or at the surface is given by

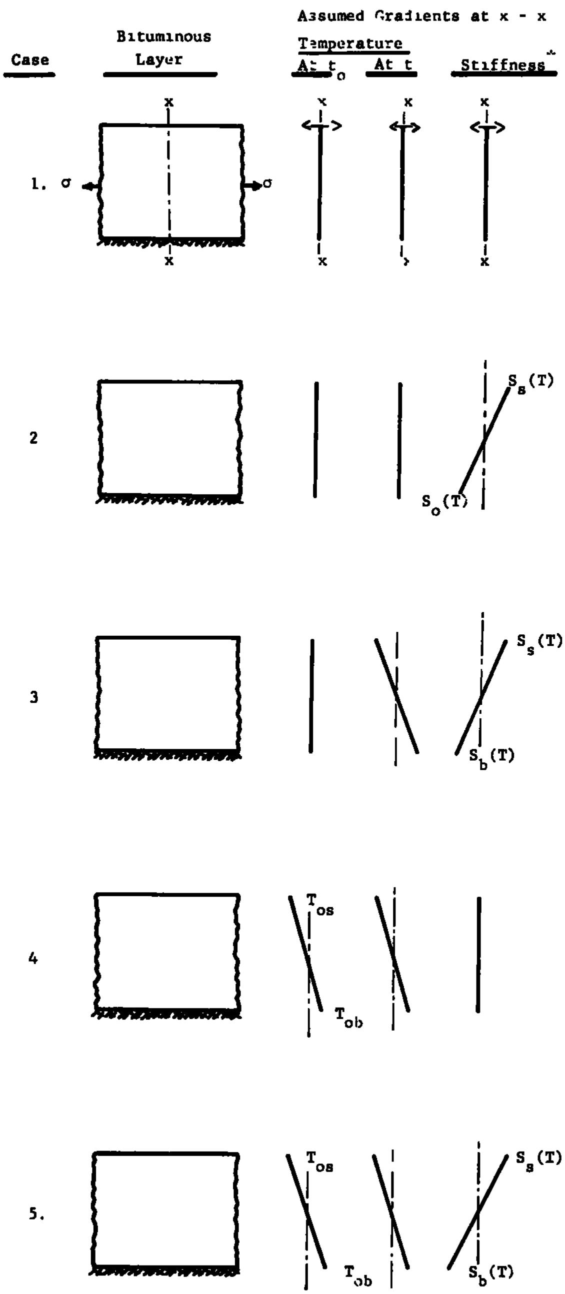
$$\sigma_x(T) = \alpha \int_{T_o}^{T_f} S(r, T)\, dT$$

or

$$\sigma_x(T) = \alpha \sum_{T_o}^{T_f} S(\Delta T)\,[\Delta T] \quad \text{as an approximation}$$

where

α = average coefficient of thermal contraction over the temperature range;
$S(r, T)$ = temperature and time rate of temperature change dependent stiffness modulus;
$S(\Delta T)$ = stiffness modulus determined at the midpoint of ΔT and using a loading time corresponding to the time interval for the ΔT change;
ΔT = finite temperature interval between T_o and T_f;



Stresses

Case 1.

Top

$$\sigma_{xs_1}(\Delta T_1) = \alpha S(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xs}(T) = \sum_{T_o}^{T_f} S(\Delta T)[\Delta T]$$

Bottom:

$$\sigma_{xb_1}(\Delta T_1) = \alpha S(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_f} S(\Delta T)[\Delta T]$$

Case 2

Top:

$$\sigma_{xs_1}(\Delta T_1) = \alpha S_s(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xs}(\Delta T) = \alpha \sum_{T_o}^{T_f} S_s(\Delta T)[\Delta T]$$

Bottom:

$$\sigma_{xb_1}(\Delta T_1) = \alpha S_b(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_f} S_b(\Delta T)[\Delta T]$$

Case 3

Top:

$$\sigma_{xs_1}(\Delta T_1) = \alpha S_s(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xs}(T) = \alpha \sum_{T_o}^{T_{fs}} S_s(\Delta T)[\Delta T]$$

Bottom:

$$\sigma_{xb_1}(\Delta T_1) = \alpha S_b(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_{fb}} S_b(\Delta T)[\Delta T]$$

Case 4

Top:

$$\sigma_{xs_1}(\Delta T_1) = \alpha S(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xs}(T) = \alpha \sum_{T_{os}}^{T_{fs}} S(\Delta T)[\Delta T]$$

Bottom:

$$\sigma_{xb_1}(\Delta T_1) = \alpha S(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_{ob}}^{T_{fb}} S(\Delta T)[\Delta T]$$

Case 5.

Top:

$$\sigma_{xs_1}(\Delta T_1) = \alpha S_s(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xs}(T) = \alpha \sum_{T_{os}}^{T_{fs}} S_s(\Delta T)[\Delta T]$$

Bottom:

$$\sigma_{xb_1}(\Delta T_1) = \alpha S_b(\Delta T_1)[\Delta T_1]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_{ob}}^{T_{fb}} S_b(\Delta T)[\Delta T]$$

* Stiffness gradients shown are exclusive of stiffness gradient induced by temp. gradient

Figure 4. Approximate method for determining thermal stresses in a bituminous pavement surface.

( ) = as a function of; and
[ ] = multiplication.

Case 2: The beam is subjected to a uniform temperature drop, with no temperature variation with depth (as in Case 1), from T_o to T_f, with the stiffness modulus, $S(\Delta T)$, varying linearly through the depth (i.e., due to asphalt hardening). For this case, the maximum thermal stresses at the surface of the layer, $\sigma_{xs}(T)$, and at the bottom of the layer, $\sigma_{xb}(T)$, are given by

$$\sigma_{xs}(T) = \alpha \sum_{T_o}^{T_f} S_s(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_f} S_b(\Delta T)\,[\Delta T]$$

where

$S_s(\Delta T)$ = stiffness modulus at the surface of the layer at any temperature, T; and
$S_b(\Delta T)$ = stiffness modulus at the bottom of the layer at any temperature T.

Case 3: The beam initially has no temperature gradient but then is subjected to a temperature drop, with the rate of decrease, dT/dt, being greater at the surface than at the bottom, so that $T_o - T_{fs}$ represents the total temperature change at the surface and $T_o - T_{fb}$ the total temperature change at the base, during the total time interval. Additionally, the stiffness modulus varies linearly through the depth (as in Case 2). Then the maximum thermally induced stresses at the surface and bottom of the layer are given by, respectively

$$\sigma_{xs}(T) = \alpha \sum_{T_o}^{T_{fs}} S_s(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_{fb}} S_b(\Delta T)\,[\Delta T]$$

Case 3a: The situation is the same as in Case 3, except that there is no stiffness gradient. Here, the maximum thermally induced stresses at the top and bottom, respectively, are

$$\sigma_{xs}(T) = \alpha \sum_{T_o}^{T_{fs}} S(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_{fb}} S(\Delta T)\,[\Delta T]$$

Case 4: The beam has an initial, linear temperature gradient from T_{os} at the surface to T_{ob} at the base. It is then subjected to a temperature drop, with the rate of decrease, dT/dt, being greater at the surface than at the bottom, so that $T_{os} - T_{fs}$ represents the total temperature change at the surface and $T_{ob} - T_{fb}$ the total temperature change at the base. For this case, the maximum top and bottom thermally induced stresses, respectively, are

$$\sigma_{xs}(T) = \alpha \sum_{T_{os}}^{T_{fs}} S(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_{ob}}^{T_{fb}} S(\Delta T)\,[\Delta T]$$

Case 5: The situation is the same as Case 4, except that the stiffness modulus varies linearly with depth (as in Cases 2 and 3). Here the maximum thermally induced stresses at the top and bottom of the layer are given by, respectively,

$$\sigma_{xs}(T) = \alpha \sum_{T_{os}}^{T_{fs}} S_s(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_{ob}}^{T_{fb}} S_b(\Delta T)\,[\Delta T]$$

MECHANISMS OF LOW-TEMPERATURE CRACK INCEPTION AND PROPAGATION

Field Observations

The incidence of low-temperature cracking has been visually observed in many instances and some years ago, Baskin (22) noted: "[I]t was repeatedly noticed that cracking does not necessarily occur during the time of year when the pavement is at its lowest temperature. Rather is cracking most noticeable during the spring when pavement temperatures range all the way from 25 F to 45 F. Time and again we find pavements subjected to temperatures of -30 F or -40 F—and clear of snow—showing hardly any cracks." The same phenomenon has been observed by many others, as typified by one of the findings of the recent Saskatchewan study (3) which noted "transverse cracking became evident after prolonged cold temperatures followed by a sudden rise in temperature rather than after prolonged cold temperatures alone."

However, there has recently been some opinion, including that of the authors of this paper, that low-temperature cracks may actually occur as very fine or micro-cracks during the cold weather and that as warming occurs, these cracks open up and become visually apparent. The validity of this should be known when published results are available from several research projects that include the installation of electrical continuity strips for measuring the time of cracking (two known projects of this type are currently under way in Alberta and in Manitoba). Previously, it had been thought, as pointed out by Shields and Anderson (23), that several possible thermal reactions could occur to result in cracking:

1. Simple thermal contraction of the surface;
2. Base course restraint to contraction of the surface;
3. Sudden warming and subsequent weakening of a highly stressed surface;
4. Shrinkage cracking of the subgrade and subsequent reflection cracking through to the surface layer.

It may be noted that all these reactions seem to be consistent with the observed phenomena; however, some inconsistencies appear in relation to the hypothesis of microcracking at cold temperatures. These will be subsequently considered in more detail since an adequate understanding of the pertinent mechanisms involved is essential to the eventual development of control or predictive techniques relating to fracture susceptibility.

A Consideration of the Cracking Mechanism

Cracking of a bituminous surface layer will occur when the induced stresses, either externally applied or internally developed, exceed the tensile strength of the material. The externally applied stresses can occur either due to traffic or to "drag" from subgrade cracking while the internally developed stresses are thought to be primarily associated with temperature changes.

It has been thought by a number of people that the visual observation of cracks at higher temperatures, following cold weather, meant that the warm weather weakened the highly stressed bituminous layer. The consequence was thought to be an insufficient tensile strength in the material to withstand the thermally induced stress from the lower temperature. This is supported by an actual observation by Huculak (24), who reported that a pavement cracked audibly beneath his feet during the sudden warming of a chinook in Banff, Alberta. However, examination of this hypothesis in the light of what is physically possible and what has been observed, reveals several contradictions. Firstly, the hypothesis assumes that the cold base course is restraining the surface course, to maintain a high stress condition, while the surface of the layer suddenly warms and thereby is insufficiently strong to resist this imposed drag. But if the subgrade and base do not crack, they obviously have not contracted in a longitudinal direction and consequently are unable to impose drag stresses on the bituminous layer. Secondly, it seems that relaxation mechanisms may be able to act sufficiently fast to relieve any such imposed stresses, especially at the 40 F to 50 F temperatures that can occur in western Canada under rapid chinook wind warming. To check this second aspect, some recent testing of asphalt-concrete beams at the University of Waterloo has shown very high thermally induced stresses in a completely restrained beam. Subsequent very rapid warming of such specimens, while still completely restrained, resulted in no cracking and rapid relaxation of stresses. These results (Fig. 5) indicate that stresses can be relieved at a warming rate much exceeding that likely in field service. These experiments are admittedly limited in scope insofar as asphalt aging, asphalt type, aggregate influence, and certain other factors are concerned.

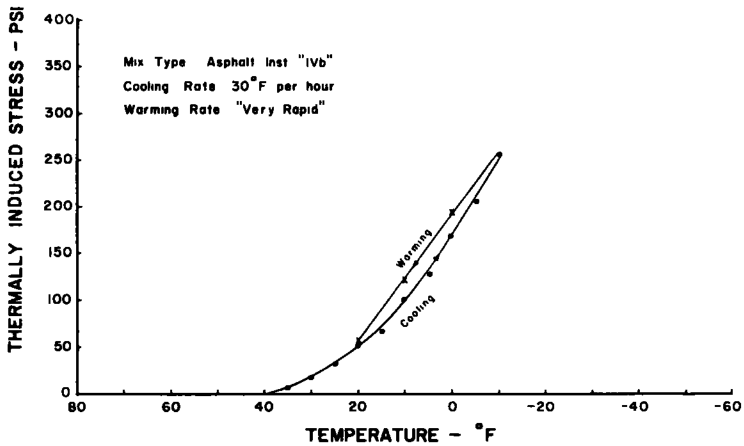


Figure 5. Thermally induced stress in a restrained asphalt-concrete beam.

Nevertheless, they tend not to support the postulated mechanism of cracking due to restraint and warming.

An Alternate Thermal Cracking Hypothesis

Any criticisms of postulated mechanisms, or advancement of new theories, must take into account the much observed fact that thermally associated cracks seem to appear primarily during the warmer weather. In other words, the cracks open up enough to be easily seen. Now if one does not accept the restraint cracking hypothesis (providing the cracking is not associated with traffic or subgrade cracks), then one might examine the fine cracking or micro-cracking postulation. This would contend that cracking does in fact occur during the cold weather, that these cracks are nearly invisible to the naked eye and that subsequent warming opens the crack. Yet, consideration of the properties of materials and the mechanics of the situation reveals a definite inconsistency. Simply, this is that if such a micro-crack did form in the bituminous layer, subsequent warming would, because of expansion, tend to close the crack rather than to open it. As well, if the crack occurred to full depth during the cold weather, it should tend to be V-shaped with a clearly visible opening at the top.

Thus, if we wish to accept the micro-cracking hypothesis, and if we accept the field observations of crack opening on warming, then we are required to formulate a new hypothesis consistent with observed phenomena. Figures 6 and 7 are schematic representations of two such possible mechanisms. They describe crack initiation and propagation in a finite series of steps, for ease of explanation, and also show the likely temperature and stress distribution patterns. The figures are fairly self-explanatory and do not require detailed discussion to illustrate the hypothesis involved. Basically, each mechanism postulates that the thermally induced crack initiates at the surface and that it penetrates to only a limited depth. Mechanism A (Fig. 6) theorizes that the crack is propagated through a stress imbalance created by warming at the surface while the remainder of the layer stays relatively cool. Then, after full-depth propagation, while the layer is still relatively cool, the crack appears visibly opened. As warming eventually spreads through the full depth of the layer (i.e., late spring or summer), the crack again closes because of expansion. This explanation of crack propagation does not seem inconsistent with Huculak's observation (24) in that he may have been witness to an extreme case of the postulated mechanism A.

Mechanism B (Fig. 7) postulates that the crack is propagated through a stress imbalance created by several cycles of day warming and night cooling, as one might expect in the spring. Then, as for the first case, the crack appears visibly opened at full-depth propagation and while the layer is still relatively cool, and closes when full-depth warming occurs.

The significant feature of both these postulated mechanisms is that the initial crack occurs to only a limited depth. This initial penetration may be very small, depending upon the temperature and stiffness gradients through the surface layer. In any case, it should be possible to check experimentally the validity of the hypothesis either in the field or in the laboratory.

Some Practical Implications

Anderson and Hahn (14) have approached the problem of thermal cracking from the point of laboratory evaluation of mixture designs, and quite logically they conclude: "It is expected that failure strain be considered as another test value to be determined, much the same as stability, flow, air voids, etc., are now obtained, prior to establishing the recommended combination of aggregate and asphalt, i.e., selection of mix design." This philosophy fits quite well with the crack initiation hypothesis advanced in that if such cracking does indeed occur to a limited depth during the cold weather, it may be possible to design the surface layer to avoid entirely or to at least reduce the cracking. The binder or leveling course, or the lower portion of a deep-strength bituminous pavement, may not require such modifications if, through the appropriate design modifications, cracking is prevented at the surface.

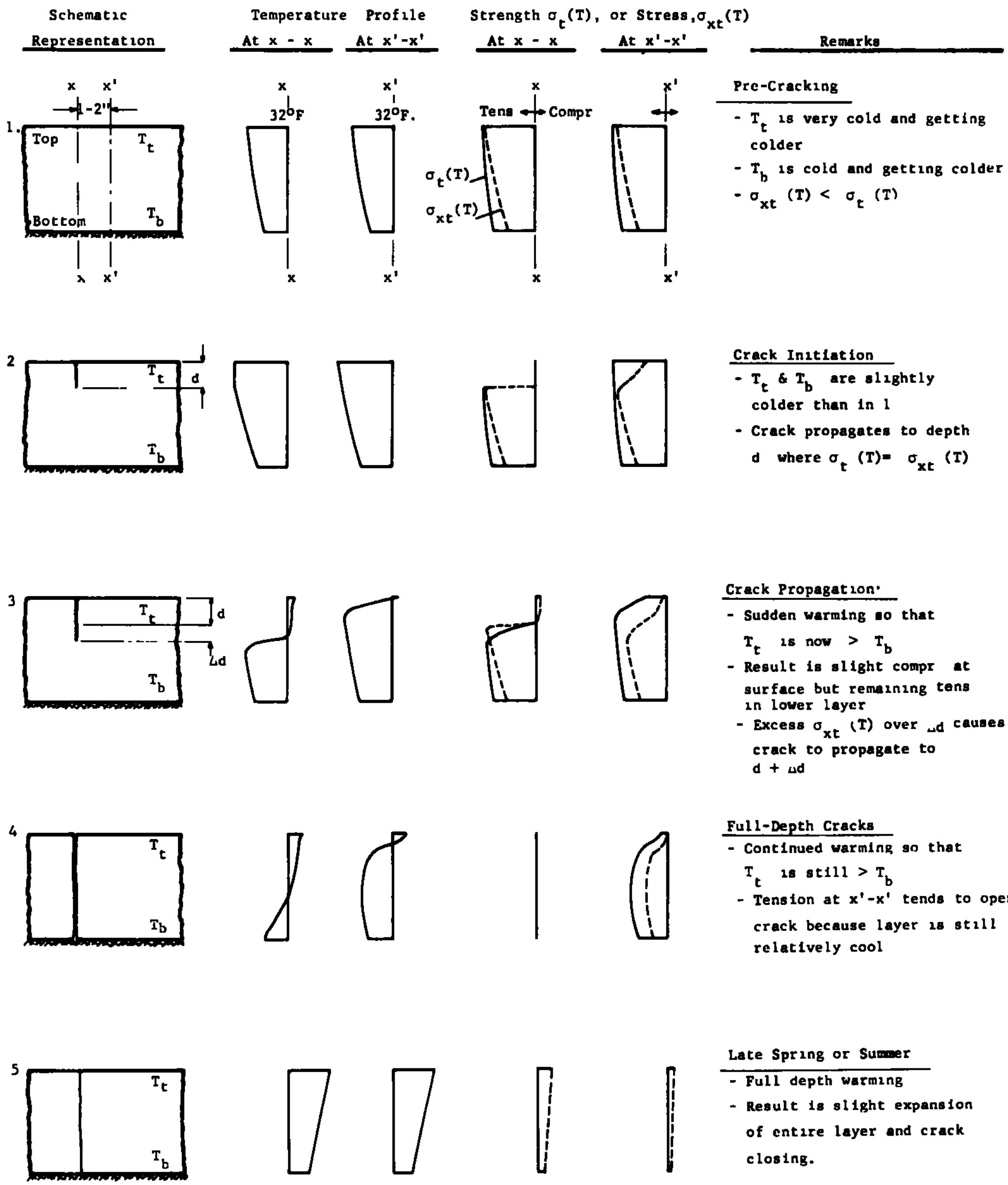


Figure 6. Possible mechanism A for low-temperature crack initiation and propagation in a bituminous pavement surface.

The first step required in such design is to evaluate the expected thermal conditions for winter conditions. It may be feasible to handle expected extreme low temperatures, for design purposes, on a return period or recurrence interval basis similar to that employed in hydrologic applications. Data from such studies as that conducted at the Clarkson Institute of Technology (21) can be most significant in providing the required temperature information. This information can then be used, with the appropriate laboratory data on tensile stress-strain-time-temperature-age characteristics, in calculating an expected thermally induced stress field in the bituminous layer for the design return period. Comparison with failure characteristics of the mixture can then be used to determine probable fracture.

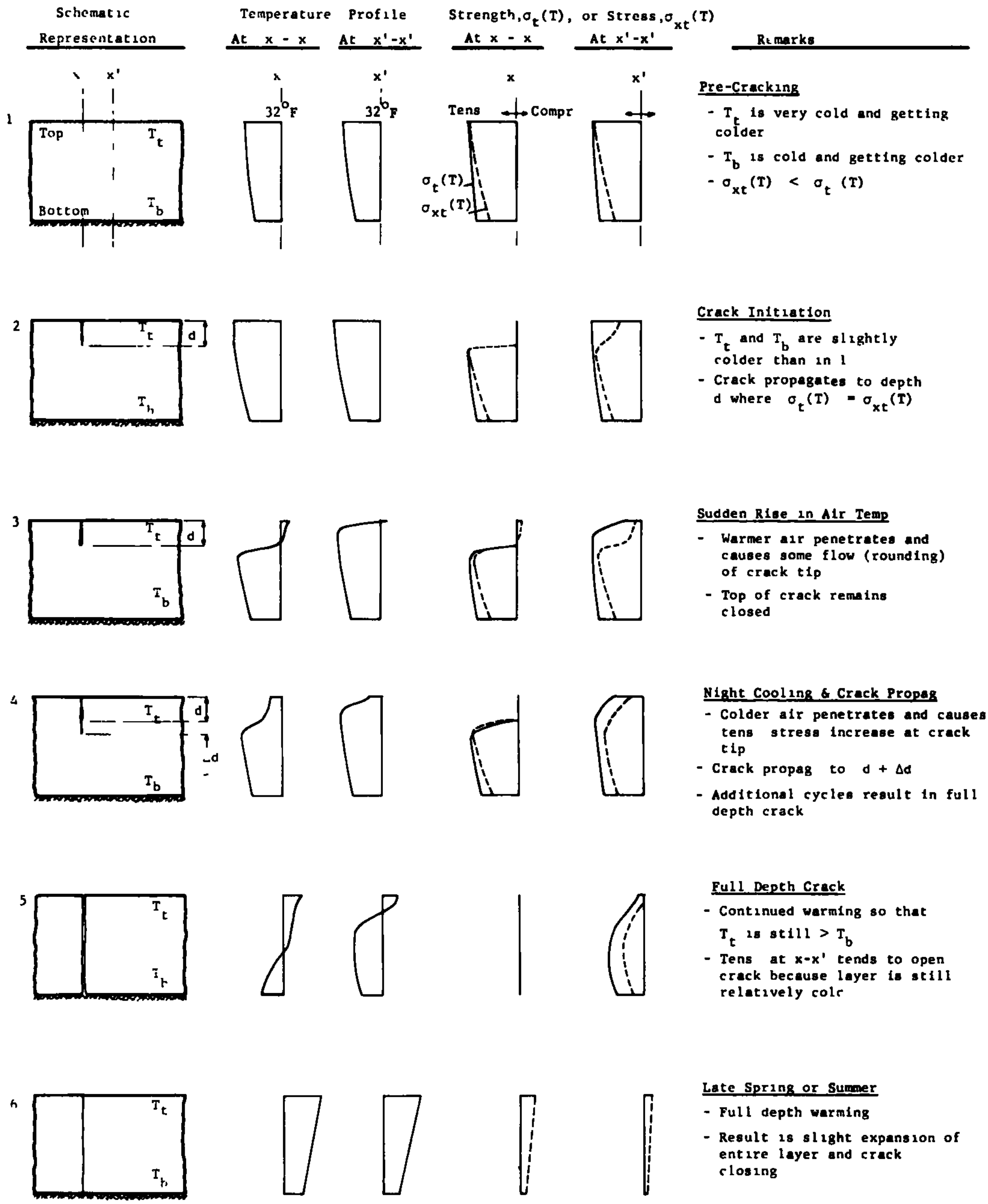


Figure 7. Possible mechanism B for low-temperature crack initiation and propagation in a bituminous pavement surface.

Another extremely important implication of the hypothesis advanced concerns the source of asphalt supply. Evidence is now being accumulated (2, 3, 14) that low-temperature transverse cracking frequency can be markedly affected by the asphalt, although it should be noted that McLeod (25) contends softer asphalt cements can reduce transverse cracking, with source apparently being a minor consideration. Nevertheless, there is a widespread opinion in Canada that we have "poor" and "good" asphalts, with regard to a variety of desirable properties including the one of reduced transverse cracking. (The terms "poor" and "good" may be far from satisfactory but find widespread usage in the industry in lieu of the availability of more objective descriptions.) Therefore, until such time as the state of technology has advanced to the point where refinery processing or additive addition can mitigate such problems as cracking, whatever the asphalt source, it may be wise to examine the potential role of the better asphalts. Since there is also considerable opinion that the proven supplies of good asphalts in Canada are limited, perhaps we are collectively guilty of some misuse. It is possible that the use of a superior asphalt for either the surface course or the top portion of a deep strength layer, where practical, may avoid low-temperature transverse cracking, providing the hypothesis of limited depth initial cracking is valid. This assumes, of course, that the temperature and stiffness gradients through the layer are such that the underlying mixtures, with the lower quality asphalts, are not overstressed.

It is, of course, realized that this implies the explicit recognition of premium grade asphalts, a practice which seems to have been largely avoided in Canada in a formal sense. Nevertheless, there are some agencies who acomplish this in a de facto sense, with their specifications. Yet, it seems that we may be wiser to extend such explicit recognition, through the appropriate specifications based on realistic physical tests, with much the same manner of reasoning that has traditionally led us to use higher quality aggregates in the higher portions of the total pavement structure. Such practices could tend to conserve the supplies of higher quality asphalts or to use them in a more efficient manner.

While the foregoing suggestions are based on limited evidence, they are considered to have sufficient promise to encourage the development of experimental programs concerned with the efficacy of using good and poor asphalts, in certain cases, in the same pavement structure. The potential payoff could be a more efficient and economical use of available materials as well as the economic benefits of reduced thermal cracking. In an attempt to answer some of these questions and in view of this potential payoff, the University of Waterloo has under way a sponsored research program in the areas of flexible pavement performance at low temperatures and in systems analysis of pavement design processes.

CONCLUSIONS AND RECOMMENDATIONS

The major conclusions arising from this paper and some pertinent suggestions may be summarized as follows:

1. An hypothesis has been advanced to the effect that low-temperature cracking of bituminous surfaces occurs in two main phases: these consist essentially of limited depth crack initiation, and subsequent full-depth propagation with warmer air and surface temperatures. Diagrams are presented to explain the pertinent mechanics and it is shown that this crack mechanism hypothesis is not inconsistent with observed phenomena. As well, certain inconsistencies between observed phenomena and previously advanced postulations are explored.
2. Low-temperature effects on pavement performance are discussed and the need for modifying current bituminous mixture design procedures to account for low-temperature service conditions is pointed out. The need for investigating the possible effects of varying degrees of asphalt aging with depth is also noted.
3. A procedure is presented for calculating thermally induced stresses in bituminous surfaces. It is approximate and can utilize stiffness moduli from experimental evaluation of the materials. The procedure recognizes the stiffness and temperature gradients that are likely present in field service and it explores a variety of possible situations. The validity of this approach has recently been demonstrated (26).
4. Certain practical implications of the cracking hypothesis advanced in this paper are discussed. These relate to reduced cracking through mix designs and through the use of good and poor asphalts in the same pavement structure. The application of such practices, which of course depends upon verification of the hypothesis, has the potential of greater efficiency or economy in the use of existing materials and in the economic

benefits of reduced cracking. Experimental programs to explore these implications seem warranted.

ACKNOWLEDGMENTS

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Pavement Cracking: Causes and Some Preventive Measures

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The pavement cracking problem was investigated with respect to deep seated movements and also from the standpoint of "shrinkage cracking" due to the inherent properties of road-building materials. In regard to the latter part of the report, some laboratory tests involving migration of lime into dried and cracked specimens were performed. Results indicated that the use of the proper percent solids in lime slurry, re-cycling of treatment and reconsolidation all contribute greatly to the effectiveness of lime's outstanding ability to reduce volume change and increase strength in an unmixed cracked clay.

This report establishes a theory that the degree of shrinkage cracking may be a function of certain strength characteristics of base and paving materials. The data for truly flexible base materials show the compressive strengths of such materials to be many times greater than are their respective tensile strengths.

A shear diagram classification chart divides all soil materials, with or without stabilization, into three groups, two of which are susceptible to shrinkage cracking and one which is not.

A chart shows the relation of compressive strength to the ratio of compressive to tensile strength for materials with widely varying characteristics. A line is drawn which appears to separate materials believed to be highly susceptible to shrinkage cracking from those which are less susceptible.

Recommendations are made for the use of water and lime to prevent deep seated swelling, adequate thickness of base to support loads, wide shoulders to prevent cracking of clay subgrades and stabilized materials which have relatively high ratios of compressive to tensile strengths. The use of high-grade flexible base materials containing low amounts of fine silt is recommended for use in construction of flexible pavements in areas susceptible to frost damage.

•THERE are many factors which contribute to the occurrence of cracks in pavements, some of which follow:

1. Excessive load deflections including resilience deflections.
2. Subsidence, consolidation, slides, etc.
3. Swell-shrink conditions of the subgrade.
4. Shrinkage cracking of the base and/or pavement due to causes other than deflections or movements of subgrade.
5. Frost and/or freeze-thaw action.
6. Brittleness of pavement due to aging and/or absence of traffic, including use of hard asphalts and their hardening due to oxidation. Type of mix and/or construction procedures may contribute to oxidation.
7. Thermal expansion and contraction.

It is intended that this report will concentrate primarily on items 3 and 4, including volume change of subgrade and "shrinkage cracking" caused by the inherent properties of the materials in the base and pavement. This does not mean that these items are the only important factors involved because cracking may be caused by any one or a combination of the items. The author has written reports indicating how triaxial tests can be used to help prevent excessive load deflections (1, 2, 3, 4, 5) and other reports have been presented in an attempt to provide for less detriment from shrink-swell conditions (6, 7, and 8). Data given in these reports will not be repeated here except to state that control of subgrade moisture content at time of evaporation cutoff, use of thick flexible base or stabilized layers consisting of wide blanket sections have done much to reduce pavement cracking but we have not eliminated the problem. Perhaps the damage from freeze-thaw or frost to some flexible base materials can be about as serious in certain portions of Texas as any other type of cracking. This is particularly true when many freeze-thaw cycles occur during wet winter months.

In areas adversely affected, the pavement cracks into blocks and base material fines pump from beneath the surfacing. An investigation involving properties of minus No. 40 materials (too numerous to include in this report) from roads noted to have various degrees of frost susceptibility indicates the following:

1. That the maximum amount of minus No. 200 in the total material often is not indicative of freeze-thaw susceptibility; it has been found to be of considerable value when expressed as a percentage of the minus No. 40 material.
2. That the percent minus 0.005-mm material expressed as a percentage of the minus No. 40 material also shows promise of correlating with performance as affected by freeze damage susceptibility unless surfacings consist of surface treatment applications.
3. Although no one single requirement placed in specifications is going to solve this problem, it now appears that in cases where surfacings are to consist of premix or HMAC, the minus No. 40 portion of unstabilized base materials produced in frost or freeze damage areas of Texas should not contain more than 25 percent minus 0.005-mm sizes nor more than 55 percent passing the No. 200 sieve.

Base materials used in such areas should be durable, have high triaxial strengths and should contain small amounts of fine size particles. Unless a thorough understanding of PVR (Potential Vertical Rise) exists, it is difficult to know when "deep treatments" are necessary to prevent cracking. Considerable success has been obtained by ponding and sealing to where PVR does not exceed ½ in., but the ponding method is so time-consuming that it has not proved to be very popular for highway construction.

CLASSES OF MATERIAL—SUSCEPTIBILITY TO CRACKING

In order to form a general concept about soil materials, the Mohr diagram classification chart (Fig. 1, see AASHO T 212) is used as a means for dividing all soil materials into the following three groups:

1. Group I generally includes triaxial classes 4, 5, and 6. This group consists of yielding materials in which the application of increasing increments of normal stresses will not be accompanied by corresponding equal increments of shearing strength.
2. Group II also consists of yielding materials but they differ from Group I in that increasing increments of normal stresses are accompanied by increments of shearing strength which are greater than the increments of normal stress applied. Generally this group includes the base and subbase materials generally used in highway work which give little trouble from shrinkage cracking.
3. Group III materials consist of cemented materials which have sufficient cohesion to form slabs. This group also has high shearing strengths but slab cracking occurs due to lack of sufficient cohesion to resist shrinkage and/or load stress. For instance, Sowers and Vesic (9) have reported tensile stresses up to 60 psi at the bottom of soil-cement slabs. Not very many stabilized soil slabs could be expected to be strong enough to resist such stresses. One example might be portland cement concrete con-

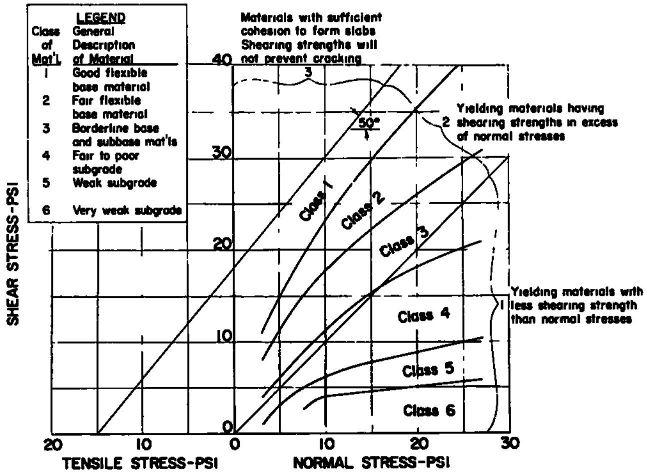

Figure 1. Mohr diagram classification chart.

tinuously reinforced to reduce severity of cracking due to additional cohesive strength being supplied by the steel. This group of slab forming materials can be expected to be more or less susceptible to shrinkage cracking regardless of quality of subgrade. It appeared that tensile and compressive stresses and/or strengths might be pertinent to the problem.

RATIO OF COMPRESSIVE TO TENSILE STRENGTHS

In order to contribute to the subject of shrinkage cracking of pavements, it became necessary for the author to conceive of theoretical as well as practical aspects of the problem. By use of the Mohr diagram of shearing stresses, certain theoretical concepts appear to be logical. When materials are capable of forming slabs (strengths sufficient to resist applied stresses), it is necessary that sufficient cohesion exist in order to resist tensile cracking. Figure 2 shows how a given amount of cohesion may

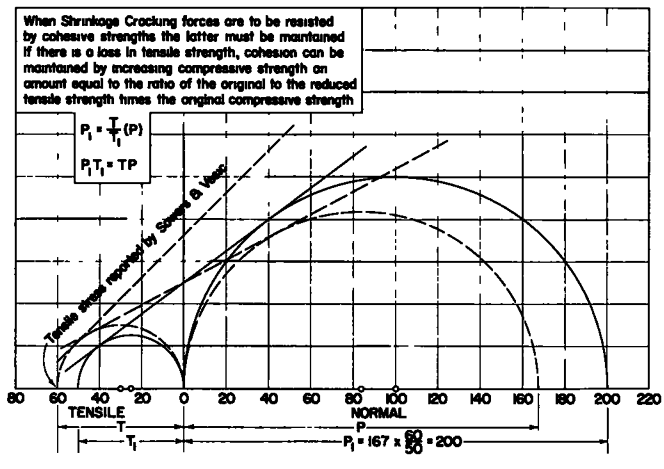

Figure 2.

be maintained by varying compressive and tensile strength relations or ratios in which

$$P_1 T_1 = PT$$

where
- P = original compressive strength,
- T = original tensile strength,
- P_1 = new or increased compressive strength, and
- T_1 = new or decreased tensile strength.

From this standpoint, it seemed that the ratio of compressive to tensile strength, hereafter referred to as CTR, would be an interesting tool for use in analyzing this problem provided we could investigate the susceptibility to shrinkage cracking of a wide variety of materials. It was soon discovered that materials with widely varying strengths would have the same CTR ratios such as steel and soil, however, they had widely different compressive strengths so it was decided to separate such materials by plotting compressive strengths against the ratio of compressive to tensile strengths (Fig. 3).

CTR RATIOS FOR A WIDE VARIETY OF MATERIALS

Figure 3 shows some correlation between compressive strength and the ratio of compressive strength to tensile strength for a number of materials having a wide variety of physical characteristics, such as steel, portland cement concrete, epoxy-sand admixtures, soil-cement mixtures, soil-lime mixtures, raw flexible base materials, a gumbo clay soil and a sand soil plus OA-90 asphalt mixture. A line is drawn which

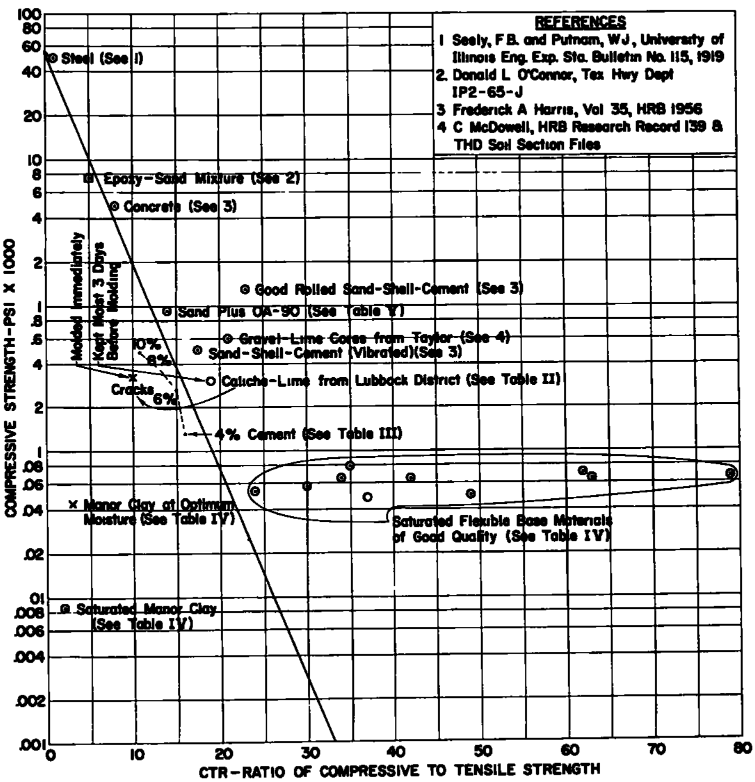

Figure 3. Relation of compressive strength to CTR.

tends to separate the mixtures on the left which are susceptible to shrinkage cracking from those on the right which are less susceptible to shrinkage cracking. Tensile strengths were determined by use of cohesiometer (converted to psi) for all materials except steel, PC concrete and epoxy-sand admixtures which were tested in tension.

TABLE 1

SOIL CONSTANTS AND GRADATION OF FLEXIBLE BASES AND SOILS

Lab No	LL	PI	SL	LS	SR	Soil Binder	W B M (% loss)
63-282-R	21	7	14	4 4	1 92	17	33
64-459-R	19	5	14	3 7	1 93	21	32
65-67-R	30	9	20	5 0	1 68	29	37
65-68-R	28	11	17	6 3	1 78	20	32
65-100-R	24	9	15	4 7	1 89	14	19
66-48-R	24	5	16	4 4	1 79	24	33
66-49-R	22	6	15	3 7	1 86	20	35
66-50-R	28	6	21	3 5	1 57	22	34
66-169-R	33	8	24	4 4	1 60	19	35
66-171-R	21	6	16	3 3	1 86	21	32
62-375-E	21	3	16	2 4	1 66	97	
64-526-R	29	14	18	5 9	1 74	42	
66-248-R	25	5	21	2 3	1 69	96	
Manor clay	70	41	11	20 0	1 93	100	

Percent Retained On Square Mesh Sieves

Lab No	Opening (in)					Sieve Numbers							Grain Diam (mm)			Spec Grav	Material
	1¾	1¼	⅞	⅝	⅜	4	10	20	40	60	100	200	0 05	0 005	0 001		
63-282-R	0	9	28	40	57	70	76	80	83	85	87	89	90	93	98	2 70	Flexible base
64-459-R	0	6	20	33	46	59	68	75	79	81	83	85	87	95	98	2 72	Flexible base
65-67-R	0	5	18	29	42	54	63	68	71	73	75	80	85	94	96	2 67	Flexible base
65-68-R	0	12	27	37	49	61	71	76	80	82	84	86	88	94	97	2 68	Flexible base
66-100-R	0	10	19	31	46	62	73	82	86	88	90	92	93	97	99	2 76	Flexible base
66-48-R	0	10	30	42	52	62	70	74	76	78	82	90	90	96	99	2 70	Flexible base
66-49-R	0	3	15	26	40	55	67	76	80	81	82	83	87	94	98	2 63	Flexible base
66-50-R	0	5	17	28	42	56	68	74	78	82	86	90	96	97	99	2 64	Flexible base
66-169-R	0	4	14	26	41	54	69	77	81	83	86	88	89	97	98	2 71	Flexible base
66-171-R	0	3	13	24	37	53	64	74	79	82	84	86	87	93	97	2 74	Flexible base
62-375-E								0	3	13	47	81	87	94	96	2 64	Sand
64-526-R	0	10	22	29	34	42	50	55	58	61	70	80	81	89	94	2 63	Flexible base
66-248-R							0	1	4	25	68	77	79	84	87	2 67	Subgrade soil
Manor clay									0	1	4	8	9	45	59	2 71	Subgrade soil

TABLE 2

COMPRESSION AND COHESIOMETER TEST RATIO

Sample No 64-526-R	Percent Stabilizer (lime)	Curing Time (days)	Compressive Strength (psi)	Dry Density (pcf)	Cohesiometer Value (psi)	Ratio (CTR)	Average Ratio (CTR)
4	2	21	336 9	120 7		10 8	10 8
1	2	21		121 2	31 1		
6	2	21	345 7	120 6		10 8	
2	2	21		121 3	32 0		
21[a]	2	21	298 2	118 8		19 0	19 1
8[a]	2	21		119 5	15 7		
25[a]	2	21	318 4	118 9		19 2	
13[a]	2	21		120 1	16 6		
11	4	21	316 5	119 2		9 6	10 2
3	4	21		120 0	33 0		
12	4	21	330 7	119 4		10 7	
4	4	21		119 8	31 0		
29[a]	4	21	311 1	117 5		14 5	14 0
9[a]	4	21		119 0	21 5		
30[a]	4	21	312 0	117 5		13 4	
10[a]	4	21		118 3	23 2		
18	6	21	320 0	117 5		14 3	14 2
5	6	21		119 9	22 4		
19	6	21	341 0	117 5		14 1	
6	6	21		119 7	24 1		
35[a]	6	21	284 5	116 8		12 7	12 8
11[a]	6	21		117 4	22 4		
37[a]	6	21	273 8	116 7		12 8	
12[a]	6	21		118 1	21 4		

[a]Moist cured 3 days before molding

Note Specimens are 6 x 8 in. molded with equipment described in AASHO T 212 and using a compactive effort of 50 ram blows per layer (10-lb segment of a circle hammer dropping 18 in)

Photograph 1. Half section of treated specimen showing lime migration between specimen cracks. Specimens clamped in metal half cylinders to facilitate slicing in half. Close-up at right shows surface of bisected lime-treated specimen after spraying with phenolphthalein. Note that darkly shaded areas are much wider than the white streaks of lime, indicating that the pH of a considerable portion of the sample has been altered.

The National Lime Association provided funds for the printing of this color photograph.

TABLE 3

COMPRESSION AND COHESIOMETER TEST RATIO

Sample No 62-375-E	Percent Stabilizer (cement)	Curing Time (days)	Compressive Strength (psi)	Dry Density (pcf)	Cohesiometer Value (psi)	Ratio (CTR)
	4	7	122 0	105 2	7 5	16 3
	6	7	259 0	107 2	18 2	14 2
	8	7	387 0	108 7	29 9	12 9
	10	7	477 0	110 2	44 0	10 8

Note Specimens are 6 x 8 in molded with equipment described in AASHO T 212 and using a compactive effort of 25 ram blows per layer (10-lb segment of a circle hammer dropping 18 in.)

The data indicate that steel and soil have similar CTR values but widely different compressive strengths. The sand admixtures containing 4, 6, 8, and 10 percent cement indicate that 6 percent may have been about the maximum of cement that should be used before shrinkage cracking becomes even more critical. The high CTR values obtained from cores and beams consisting of sand-shell-cement perhaps explain why these mixtures exhibit low amounts of shrinkage cracking in the field. Details of materials tested in this laboratory are given in Tables 1 through 5. References relative to data obtained elsewhere are shown in Figure 3.

Caliche lime materials which were notable for shrinkage cracking were also investigated with respect to moist curing 3 days before compacting. It may be noted that this procedure almost doubled the CTR value and very little compressive strength was lost due to delayed compaction.

Results on CTR values obtained by testing ten good crushed stone and/or caliche flexible base materials vary from 25 to 80 (Fig. 3). These findings are consistent with the theory that CTR points for materials exhibiting small tendency toward shrinkage cracking should fall to the right of the sloping line.

TABLE 4

COMPRESSION AND COHESIOMETER TESTS FOR MANOR CLAY AND FLEXIBLE BASE MATERIALS

Lab No of Soil	Dry Density (pcf)	Cohesiometer Value[a] (psi)	Unconfined Compressive Strength[b] (psi)	CTR Ratio[c]
Manor clay saturated	92 2	4 2	8 5	2
Manor clay at optimum	94 0	14 4	43 8	3
63-282-R	135 6	1 0	62 5	63
64-459-R	139 9	1 3	47 7	37
65-67-R	135 4	1 9	56 4	30
65-68-R	137 6	1 9	64 5	34
65-100-R	149 7	1 0	49 2	49
66-48-R	143 7	1 5	63 2	42
66-49-R	138 1	2 2	52 1	24
66-50-R	125 9	0 8	63 5	79
66-169-R	131 1	1 1	67 9	62
66-171-R	136 4	2 2	76 2	35

[a]Cohesiometer specimens molded by gyratory compactor to comparable moistures and densities in all specimens except Manor clay
[b]Unconfined compressive strengths obtained from specimens molded and tested according to AASHO T 212
[c]Nearest whole number

TABLE 5

COMPRESSION AND COHESIOMETER TESTS FOR HOT-MIX ASPHALTIC MATERIALS

Lab No of Soil	Dry Density (pcf)	Cohesiometer Value (psi)	Unconfined Compressive Strength (psi)	CTR Ratio[a]
66-248-R + 8½% OA-90	136 1	65 3[b]	937 5[c]	14

[a]Nearest whole number. [b]Average of 6 tests at 74 F [c]Tested at 40 F.

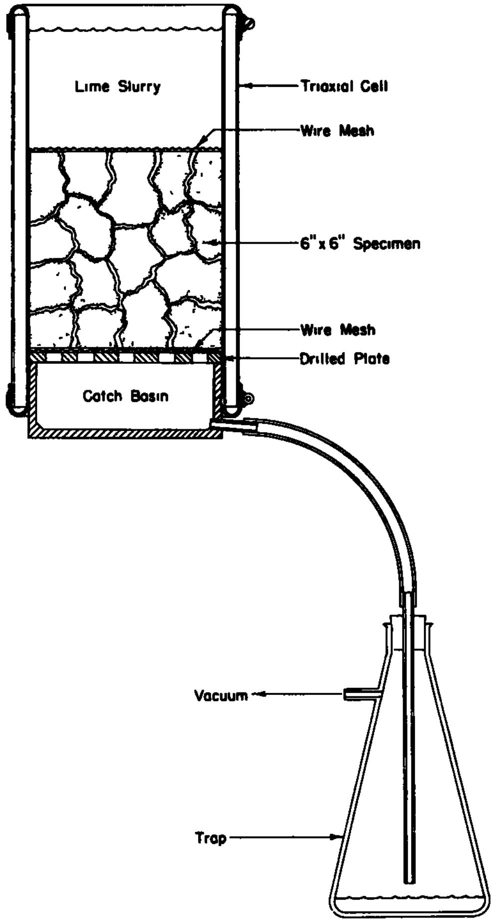

Figure 4. Lime migration apparatus.

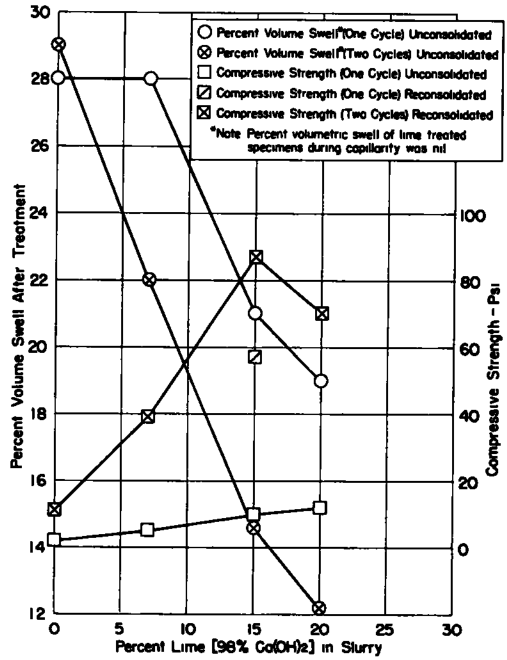

Figure 5. Lime-slurry treatment of Manor clay (gravitation method); percent volume swell and unconfined compressive strength in relation to percent lime solids in lime slurry.

Compression-tension ratios for asphaltic concrete will fall to the left of the sloping line (Fig. 3) unless tests are run at low temperatures. Results of tests run at low temperatures on one mixture of sand plus OA-90 asphalt are shown in Figure 3. Tests run on identical mixture at 140 F are plotted on the left side of the sloping line. It is doubtful if shrinkage cracking of asphaltic mixtures occurs at such elevated temperatures. Additional testing is indicated; however, if tests could be run at pertinent temperatures, there is a possibility that the suggested minimum CTR values (Fig. 3) might have application to bituminous mixtures.

MIGRATION OF LIME

Presently we hear a great deal about migration, pressure injection and deep mixing of lime for the purpose of improving volume change and strength characteristics of soils in Louisiana, Oklahoma, and Texas. Although field results indicate some success from methods, it would seem that some laboratory-scale tests might be of some assistance in evaluating proposed treatments.

In an attempt to determine the effects of lime slurry migration, some experiments using specimens consisting of "black gumbo" soil were subjected to the following techniques:

1. Specimens 6-in. high by 6-in. diameter were molded in three layers at optimum moisture for 5.3 ft-lb/cu in. compactive effort (10-lb ram, 18-in. drop).
2. Specimens were dried in a 140 F oven 96 hours.

3. Lime slurry consisting of various percentages of solids was pulled downward through the cracks by a vacuum pump (Fig. 4) and recirculated through the specimen until cracks were sealed, thereby preventing further circulation of the slurry. At this time measurements were taken to calculate percent volumetric swell.
4. Specimens were sealed in cells and moist cured for seven days.
5. Specimens were subjected to 20 days of capillarity either "as is" or after being reconsolidated to their original molding density.
6. After 20 days capillarity, specimens were measured for volume change and strength characteristics. In some instances the drying and lime slurry migration procedures were repeated before subjecting to capillarity.

Some specimens taken from step 3 were cut in half (Fig. 4) and sprayed with phenolphthalein to show the effect of lime migration on the pH of the soil. (See bands formed in Photograph 1.) Results from a number of other tests are shown in Figure 5 where percent dry lime solids in slurry is plotted as abscissa and volumetric swell (percent) and compressive strength (psi) are plotted as ordinates. The percent volumetric swell is based on the dry volume of the raw soil specimen. Calculating volumetric change on the basis of oven-dried volumes is perhaps much more severe than may be expected from field conditions. It is used in this case merely for comparative purposes and the extremely high volume change condition of the dry clay specimens was drastically reduced by using lime slurry instead of tap water. Recycling of the drying and slurry treatment procedure helps decrease swelling probably because greater amounts of lime were deposited in the specimen. The amount of lime deposited ranged from 0.9 to 2.1 percent for one cycle and 2.6 to 3.7 percent for two cycles. The amount of lime deposited and the resulting plasticity index indicated that the optimum percent of solids in the lime slurry for this type of treatment appeared to be between 15 and 20 percent.

Strength curves (Fig. 5) show that unconsolidated specimens subjected to 20 days capillarity had unconfined compressive strengths which were increased 3- to 6-fold; with reconsolidation to original molding densities, strengths were increased from 4- to 8-fold. The ability of lime to improve the quality of soils without mixing is believed to be worthy of note.

Although the module consisting of small specimens representing a fairly long section of roadway is not ideal, it is believed that the results of these experiments strongly indicate that lime pressure injection and deep mixing of lime into jointed clays of the semi-arid regions has definite possibilities of reducing volume change and increasing subgrade support.

CONCLUSIONS

The results of this investigation indicate the following conclusions to be justified:

1. The data indicate that the relation of compressive strength to compression-tensile strength ratio has an influence on shrinkage cracking of pavements.
2. The use of pressure lime injection and/or deep mixing appears to be effective for treating some jointed or cracked clays in semi-arid to arid regions for the purpose of preventing excessive volume change and strength loss of subgrade soils. The feasibility of treating soils in this manner will depend upon unit costs which are not available at this time.
3. Optimum percent of solids in lime slurry for injection purposes appears to be between 15 and 20 percent.
4. Recycling of drying and lime migration procedures reduces swelling and improves subgrade support values.
5. Reconsolidation of lime-injected soils increases supporting power of some subgrade soils greatly.

RECOMMENDATIONS

In order to prevent some of the potential hazards associated with pavement cracking, the following proposals are offered:

1. That adequate thicknesses of surfacings, bases and subbases be used so as to support the traffic loads for the life of the pavement desired.

2. That potential vertical rise be kept as low as possible, perhaps below ½-in. This may mean moisture control of clay subgrade before and after subgrade rolling operations, probably involving ponding areas where layers are capable of producing considerable amounts of volume change and covering subgrade with suitable layer capable of retarding evaporation.

3. That swell-shrink conditions of clay subgrades be controlled by use of shoulders consisting of granular and/or stabilized soils which are wide enough to control moisture fluctuations.

4. That the soil binder portion of flexible base materials to be used in frost susceptible areas of Texas should not contain more than 25 percent minus 0.005-mm material.

5. That, when feasible, all base and pavement layers be constructed out of materials whose ratio of compressive to tensile strength varies from a minimum of 11 for 1500-psi compressive strength material to a minimum of 22 for 40-psi material.

6. That highway research sections, especially in cuts, involving marls, jointed clays, etc., which have high swell potential, be treated with several cycles of lime injection prior to paving. If fairly large differential movements due to volume change of soils are anticipated, especially such as at grade points, fence lines, and old road crossings, it is recommended that serious consideration be given to use of the Oklahoma deep mixing process of lime treatment of subgrades.

ACKNOWLEDGMENTS

The author is indebted to many who have contributed and encouraged the development of this report. The work of the members of ths Soils Section and other members of the Materials and Tests Divisions of the Texas Highway Department, under the able guidance of A. W. Eatman, has been a major factor in making this report possible.

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Outline of Paper*

Reduction in Transverse Pavement Cracking By Use of Softer Asphalt Cements

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Low-temperature transverse pavement cracking is currently the most serious asphalt pavement performance problem in Canada. Analysis of samples from pavements in service and observation of their field behavior indicate that low-temperature transverse pavement cracking can be dramatically reduced by the use of softer asphalt cements. The results of theoretical studies are reported that support this conclusion. Evidence is presented, based on consideration of transverse pavement cracking in cold weather, that opposes grading asphalt cements by viscosity at 140 F, and firmly supports their continued grading by penetration at 77 F.

1. Most of the subject matter of this paper was presented at the 1968 Annual Meeting of the Highway Research Board, but was not published. Additional information, particularly on pavement samples, has been obtained since that time, and has been included in the present paper.

2. Transverse cracking is the most serious asphalt pavement performance problem in Canada between the Rocky Mountains and the mouth of the St. Lawrence River. Practically every conventional asphalt pavement made with 85/100 penetration asphalt in northern Ontario and Quebec, and those containing 150/200 penetration asphalt in the Prairie Provinces, that are more than two years old, have transverse cracks at intervals of from 5 to about 30 ft. This is true regardless of variations in the gravel aggregates being used for the paving mixtures, in the roadbed on which these pavements rest, and in the provincial highway departments' design and construction procedures. This problem of transverse pavement cracking is so serious that an answer must be found.

3. Observations of pavement performance in the field indicate that transverse pavement cracking in Canada is due to low temperature stresses and strains, since severity of pavement cracking appears to parallel severity of cold weather.

4. Looking back over the history of asphalt usage in Canada since 1930, it is recognized that with the softer SC 2, 3, 4, and 5 grades, or equivalent, employed for paving rural highways in earlier years, transverse pavement cracking was not a problem. It did not become a serious problem until after passage of the Trans-Canada Highway Act in 1949, when due to the higher standards that were adopted for the Trans-Canada Highway, the Prairie Provinces changed to 150/200 penetration, while considerable 85/100 penetration was used in northern Ontario and Quebec. Therefore, the history of asphalt usage in Canada indicates the need to return to softer grades of asphalt cement if transverse pavement cracking is to be avoided.

5. Observation of the performance of pavements in Norway that contain 300 penetration asphalt, and the Alberta Highway Department's experience with MC 2 and MC 3 mixed prime employed as a temporary pavement for from two to four years, are cited as examples of using soft asphalt cements that do not result in transverse cracking.

*This is an outline of the original manuscript; a copy of the complete paper can be obtained from the author.

6. In some milder parts of Canada, transverse cracks do not develop in pavements containing 85/100 penetration asphalt until after a number of years of service. This is believed to be due to the gradual hardening of the asphalt cement with age, and to the eventual corresponding loss of sufficient pavement flexibility to avoid transverse cracking. This provides further evidence that low-temperature transverse pavement cracking is associated with hard asphalt cements.

7. Like most generalizations, some exceptions to the conclusion that softer grades of asphalt cement would result in less transverse pavement cracking occurred on the Trans-Canada Highway north of the Great Lakes, where several pavements containing 150/200 penetration asphalt were badly cracked as well as those made with 85/100 penetration. Through the courtesy of the Ontario Department of Highways, samples were obtained from pavements in this area containing 85/100 and 150/200 penetration asphalt cements. At the same time a transverse crack survey was made of the pavement for a distance of 1000 ft at each sample location. The asphalt cements recovered from these pavement samples by our research department showed that for this particular region, very little transverse cracking occurred when the recovered asphalt had a penetration of 60 or higher at 77 F, and a penetration of 20 or more at 32 F. Consequently, for the milder low temperature conditions that occur immediately north of the Great Lakes, evidence from pavement samples and from transverse crack surveys in the field indicates that transverse pavement cracking could be reduced and largely eliminated by using softer asphalt cements, and by employing pavement design and construction procedures that would prevent hardening of the asphalt cement in service to 60 penetration at 77 F and to 20 penetration at 32 F. These penetration criteria would have to be increasingly higher for still colder areas.

8. In Manitoba, all asphalt pavements containing 150/200 penetration asphalt cements that are more than two years old have numerous transverse cracks, while those made with SC 6 (300/400 penetration) and SC 3000 (SC 5) are practically free from transverse cracks. Asphalts recovered from samples of these pavements obtained through the courtesy of the Manitoba Department of Highways showed penetrations at 77 F of 60 to 81 for the 150/200 penetration pavements, penetrations of 133 to 160 at 77 F for SC 6 (300/400 penetration) pavements that were up to 16 years old, and penetrations of 320 to 374 at 77 F for SC 3000 (SC 5) pavements that were three years old. These data provide further confirmation that the use of soft asphalt cements can dramatically reduce low temperature transverse pavement cracking.

9. The present investigation of transverse pavement cracking in Canada is related to a review of the pavement cracking problem in the central Midwest and central Atlantic Coast regions of the United States in the 1930's by The Asphalt Institute, which indicated a close relationship between pavement cracking and hardness of the asphalt binder. The results of the present study extend the conclusions of the earlier Asphalt Institute inquiry into a colder area.

10. It is recognized that the subgrade and granular base may be responsible for some transverse pavement cracking. Nevertheless, little or no transverse pavement cracking was observed when the softest grades of asphalt cement were employed. This leads to the tentative conclusion that in Canada at least, low temperature transverse pavement cracking is ordinarily due more to the characteristics of the paving mixture than to the foundation.

11. Experience has shown that little reduction in transverse pavement cracking occurs by changing to the immediately adjacent softer grade of asphalt cement, for example from 150/200 to 200/300 penetration. It is necessary to jump over one or two grades to a substantially softer grade, for example, from 85/100 to 150/200 penetration, or from 150/200 to 300/400 penetration, to achieve a very marked reduction in low temperature transverse cracking. At the same time, every good design and construction technique should be employed that will retard the rate of hardening of the asphalt cement in service. In addition to starting with a softer grade of asphalt cement, this includes designing for thicker asphalt films, low air voids, and requiring compaction by rolling during construction to much higher density, and preferably to 100 percent of laboratory compacted density.

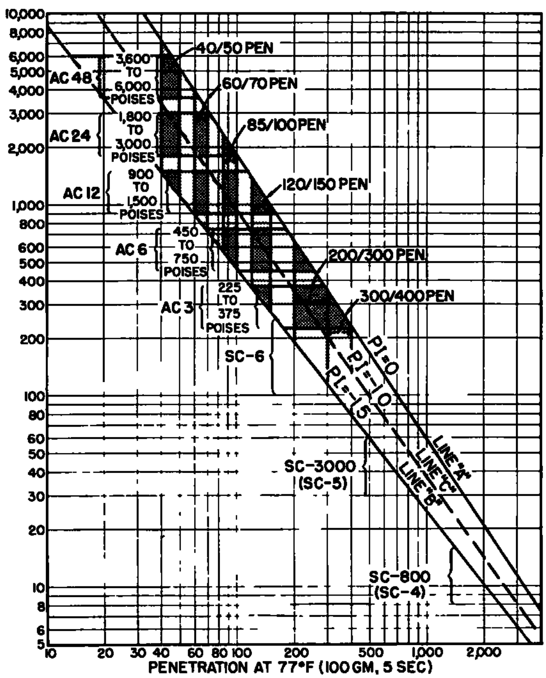

Figure 1. Correlation between viscosity at 140 F and penetration at 77 F for asphalt cements.

12. SC 6 should be discarded from all specifications in which it currently appears, because depending on crude oil source it includes all grades of asphalt cement from 150/200 to 600 penetration (Fig. 1) which in Canada, brackets lower penetration (150/200) or harder asphalt cements that result in serious transverse cracking, with higher penetration (600) or softer asphalt cements that do not. Consequently, any current SC 6 specification should be replaced with a specification for 300/400 penetration asphalt cement.

13. The adoption of softer grades of asphalt cements to reduce transverse pavement cracking could lead to two practical problems: (a) delayed rolling behind the spreader because of softness of the paving mixtures at high temperatures, and (b) rapid densification of a new pavement by traffic to 100 percent of laboratory compacted density which could lead to flushing or bleeding a few months after construction unless the paving mixture is properly designed. It is shown that the use of variable tire pressure pneumatic rollers, equipped for rapid adjustment of tire inflation pressure, would provide a solution to the first problem, and designing paving mixtures to have a minimum air voids value of 3 percent provides a simple and practical solution to the second.

14. Unlike the costly solutions proposed for most roadway problems, because soft asphalt cements are normally the same price as harder asphalt cements, the use of softer asphalt cements to solve the problem of low temperature transverse pavement cracking will not increase the initial cost of a pavement. Furthermore, there should be a substantial annual savings in the maintenance cost of filling from 100 to 300 or more cracks per mile per year.

15. Good engineering judgment is required when selecting a soft grade of asphalt cement to minimize transverse pavement cracking in cold weather, that is consistent with the need for adequate pavement stability for traffic in warm weather.

16. Since the performance of pavements in the field appeared to indicate the need for softer asphalt cements to minimize low temperature transverse pavement cracking, the next step was to look for theoretical confirmation of this conclusion.

17. Rader, in three papers published in AAPT Proceedings in the 1930's, concluded from his investigation of actual pavement samples in the laboratory, that if pavement cracking at low temperatures is to be avoided, pavements should have a low modulus of elasticity. He stated that this could be most easily achieved by using soft asphalt cements.

18. It is shown in the paper that the tensile stress induced in a pavement due to pavement contraction caused by chilling over a given low temperature range in a specified time, varies directly with the modulus of elasticity of the pavement. When the tensile stress exceeds the tensile strength of a pavement, a crack occurs.

19. Heukelom, in the 1966 AAPT Proceedings showed that the strain at which an asphalt cement cracks when chilled, decreases very rapidly with an increase in the modulus of stiffness or hardness of an asphalt cement. (Rader's modulus of elasticity

is synonymous with Heukelom's modulus of stiffness.) This means that when chilled over a given low temperature range in a stipulated period of time, a pavement containing a hard asphalt cement is more likely to crack than when it contains a soft asphalt cement.

20. Hills and Brien, for a prepared discussion in the 1966 AAPT Proceedings, employed Heukelom's method to calculate the fracture temperatures (the temperature at which a pavement cracks because the tensile stress exceeds its tensile strength) of paving mixtures when they are cooled at a specified rate to low temperature. They also checked these calculated fracture temperatures by laboratory tests on actual paving mixtures and obtained quite good agreement. Hills and Brien concluded, as illustrated by Figure 2, that the temperature at which pavement cracking is to be expected can be lowered very substantially by the use of softer asphalt cements, and that for any penetration grade of asphalt cement, a lower pavement cracking temperature is associated with a higher PI (higher viscosity) than for a lower PI (lower viscosity) paving asphalt.

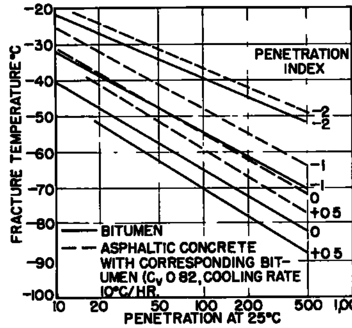

Figure 2. Influence of penetration and temperature susceptibility of bitumen on the fracture temperature of bitumen and asphaltic concrete. (Courtesy Hills and Brien.)

21. Rader's conclusions, relating modulus of elasticity (modulus of stiffness) of asphalt pavements to low temperature pavement cracking, were based on samples from particular pavements that he selected. Even if a simple laboratory test for measuring the modulus of stiffness of any proposed paving mixture can be devised, there is need for some general theoretical method that will enable an engineer to foresee or to forecast the influence that any proposed change in paving mixture design is likely to have on a paving mixture's modulus of stiffness (modulus of elasticity). Nomographs developed originally by Van der Poel and by Pfeiffer and Van Doormaal, that were modified by Heukelom and Klomp, and modified further by the author, make a general theoretical method for this purpose possible (Fig. 3). The procedure for preparing such charts is described in the paper.

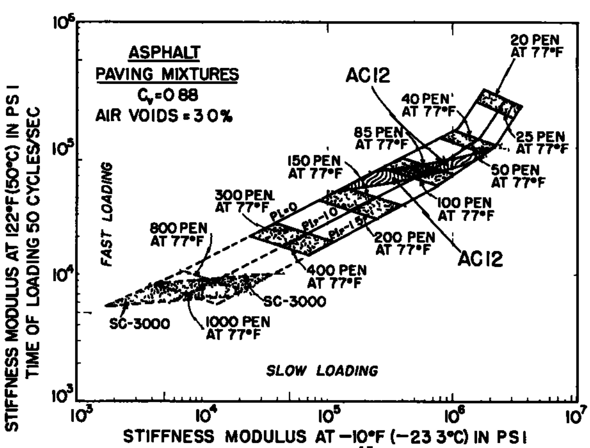

Figure 3. Relationship between stiffness moduli for asphalt paving mixtures for high rate of loading (high-speed traffic) at high temperature (122 F) vs slow speed of loading (temperature stresses) at low temperature (-10 F).

22. Figure 3 pertains to compacted paving mixtures containing 3 percent air voids, in which the volume of the aggregate is 87 percent of the volume of aggregate plus asphalt cement. These paving mixture would satisfy The Asphalt Institute design criteria for dense graded asphalt concrete made with aggregate of either ³⁄₄- or ⁵⁄₈-in. maximum particle size. The abscissa makes it possible to compare the moduli of stiffness of a specified paving mixture that have been developed by slow chilling to -10 F, when it contains 20/25, 40/50, 85/100, 150/200, 300/400 penetration asphalt cements, and SC 3000 (SC 5). Figure 3 shows that the modulus of stiffness of the paving mixture for this low temperature condition increases from about 2000 psi for the paving mixture containing SC 3000 (SC 5) with

a PI of 0.0 (high viscosity), to 2,000,000 psi when the paving mixture contains 40 penetration asphalt with a penetration index of -1.5 (low viscosity). This range of 1000-fold in modulus of stiffness is due solely to the differences in the hardness of the asphalt binder the paving mixture contains. Rader reported that the use of a softer asphalt cement was an effective method for reducing the modulus of stiffness of a paving mixture. This is verified by Figure 3.

23. Three full-scale test roads, each six miles long and each containing three 2-mi paved test sections, were constructed in southwestern Ontario in 1960 by the Ontario Department of Highways, to test the performance of asphalt pavements containing three different 85/100 penetration asphalt cements, one of high viscosity or high PI, one of intermediate viscosity or intermediate PI, and one of low viscosity or low PI. A different asphalt cement was incorporated into the pavement for each 2-mi test section, and all three asphalt cements were used in each 6-mi test road. In 1968 when these pavements were eight years old, a transverse crack survey was made, and the asphalt cement was recovered by our research department from pavement samples obtained through the courtesy of the Ontario Department of Highways from the nine test sections in the three test roads. Data on the recovered asphalt cements showed that in eight years they had in general hardened from 85/100 penetration to 30/40 penetration. Figure 4 shows a plot of the number of Type I transverse cracks per mile (transverse cracks that extend across the full lane width) vs the penetration index (PI) of the original 85/100 penetration asphalt cements. Figure 4 implies that a pavement made with a low viscosity 85/100 penetration asphalt cement with a PI of -1.5 could, after 8 years of service in southwestern Ontario, be expected to develop from 20 to 50 times as many Type I transverse cracks per mile as a pavement made with a high viscosity 85/100 penetration asphalt cement with a PI of 0.0. This confirms Hills and Brien's conclusion that for the same penetration grade of asphalt cement, more low temperature pavement cracking can be expected when the pavement contains a low PI (low viscosity) than when it contains a high PI (high viscosity) asphalt cement (Fig. 2).

24. In 1961, the Ontario Department of Highways constructed a test road 9 miles long in southwestern Ontario, to compare the pavement performance of two low viscosity (low PI) asphalt cements, one of 85/100 penetration, the other of 150/200 penetration. Both asphalt cements were from the same crude oil source, and both had a penetration index of approximately -1.64. The 85/100 penetration asphalt employed was the same as that provided by Supplier 3 (Fig. 4) for the three Ontario test roads constructed in 1960. This 9-mi test pavement was 2 in. thick and was laid as a single course, whereas the three 1960 test roads were 3 in. thick and were placed in two courses. A crack survey was made in 1967 (six years old) on representative sections of this 1961 test road just before a second course of asphalt concrete (which has been planned initially as part of stage construction) was laid. Point A (Fig. 4) indicates more Type I transverse cracks per mile in the 85/100 penetration pavement of this 1961 test road than in the worst of the three 1960 test roads. In part this may be due to the 2-in. pavement thickness of the 1961 test road as compared with the 3-in. pavement thickness of the three 1960 test roads, since when all other factors are equal, thinner pavements appear to develop more low temperature transverse cracks than thicker pavements. However, the portion of the

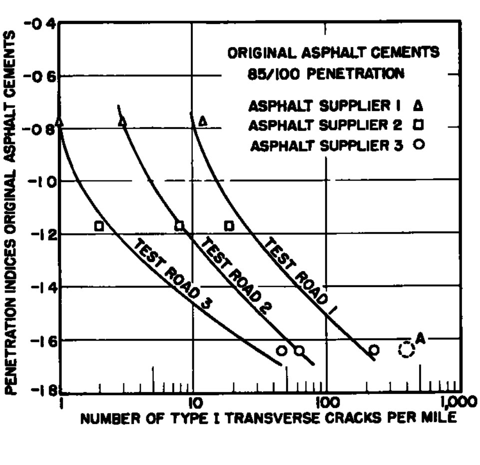

Figure 4. Relationship between penetration indices of original 85/100 penetration asphalt cements vs number of Type I transverse pavement cracks per mile after eight years of service.

1961 test road pavement containing the low PI (low viscosity) 150/200 penetration asphalt, which was also only 2 in. thick, showed no transverse cracking of any kind. That is, there was less transverse pavement cracking (actually no transverse cracking) in the pavement containing the low PI 150/200 penetration asphalt, than occurred in any test section made with 85/100 penetration asphalt with the highest PI (Fig. 4).

25. Consequently, observation of pavement performance in the field, analysis of samples from pavements in service, and the results of the theoretical studies that have been made, indicate that when all other factors are equal, low temperature transverse pavement cracking can be dramatically reduced and even eliminated by the use of softer asphalt cements. The use of softer asphalt cements for this purpose can be made still more effective, if it is combined with improved pavement design and construction procedures that will substantially retard the rate of hardening of the asphalt cement in service.

26. This study of low temperature transverse pavement cracking has a contribution to make to the current controversy over the proposed grading of asphalt cements by viscosity at 140 F with complete elimination of the penetration test, vs the current method of grading paving asphalts by penetration at 77 F. Figure 1 compares the grades AC 3, AC 6, AC 12, AC 24, and AC 48, that would result from the proposed grading by viscosity at 140 F vs the current grades in terms of penetration at 77 F. Figure 1 shows that one of the proposed grades by viscosity at 140 F, AC 12, would include all penetration grades at 77 F from 40/50 to 150/200 penetration. Consequently, grading asphalt cements by viscosity at 140 F implies with respect to AC 12, that pavement performance will be the same regardless of whether a paving mixture is made with 40/50 or with 150/200 penetration asphalt cement. It will be demonstrated that this assumption could lead to disaster as far as low temperature transverse pavement cracking is concerned.

27. The abscissa of Figure 3 indicates that the range of modulus of stiffness for slow loading at -10 F is about 4-fold for paving mixtures containing any current penetration grade, 85/100, 150/200 penetration, etc. For example, for the paving mixture containing 100 penetration asphalt with a PI of 0.0, the modulus of stiffness is 275,000 psi, but it is 1,000,000 psi when the paving mixture contains 85 penetration with a PI of -1.5. Figure 4 shows that for the 85/100 penetration grade, this 4-fold range in modulus of stiffness is associated with a range of from about 20-fold to about 50-fold in the number of low temperature transverse pavement cracks that occurred after 8 years of service in southwestern Ontario. Figure 1 indicates that for the proposed AC 12 grade for example, the corresponding range of penetration at 77 F is from 40/50 penetration to 150/200 penetration. The abscissa of Figure 3 shows that for a paving mixture containing approximately 180 penetration with a PI of 0.0, the modulus of stiffness at -10 F is about 110,000 psi, but it is 2,250,000 psi when the same paving mixture contains a 40 penetration asphalt cement with a PI of -1.5, a range of 20-fold in modulus of stiffness. Consequently, grading by penetration at 77 F provides paving mixtures with a 4-fold range in modulus of stiffness with an associated range of from 20-fold to 50-fold in the number of low temperature transverse pavement cracks per mile. Grading by viscosity at 140 F provides paving mixtures with a 20-fold range in modulus of stiffness ness, and this implies a corresponding range in degree of transverse cracking that is several times wider than from 20- to 50-fold. Paving mixtures containing AC 12 asphalt with the lowest penetration at 77 F (40/50 penetration) would show excessive low temperature transverse cracking, while those containing AC 12 with the highest penetration at 77 F (150/200 penetration) would show very much less, and possibly even no transverse cracks.

28. While grading asphalt cements by penetration at 32 F has not been proposed, Figure 5 indicates that this method for grading paving asphalts would have the great merit of eliminating the effect of differences in asphalt temperature susceptibility (PI) on modulus of stiffness values for paving mixtures subjected to slow loading by slowly chilling to -10 F. For four of the ranges of penetration at 77 F (Fig. 3), 40/50, 85/100, 150/200, and 300/400 penetration, Figure 5 demonstrates that for paving mixtures containing asphalt cements with the four corresponding ranges of penetration at 32 F, the boundaries for each grade by penetration at 32 F are vertical lines from PI = 0.0 to PI = 1.5, and the range in pavement modulus of

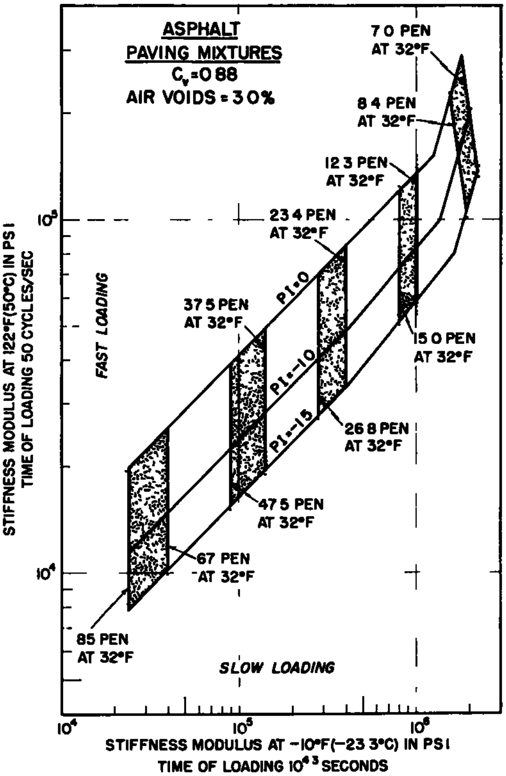

Figure 5. Relationship between stiffness moduli for asphalt paving mixtures for high rate of loading (high-speed traffic) at high temperature (122 F) vs slow speed of loading (temperature stresses) at low temperature (-10 F).

stiffness values for each grade of asphalt cement in terms of penetration at 32 F is only 1.5-fold. In comparison, when asphalts are graded by penetration at 77 F, Figure 3 shows that the range in modulus of stiffness is about 4-fold for paving mixtures containing each grade, and when asphalt cements are graded by viscosity at 140 F with elimination of the penetration test, for example AC 12, the permissible range in modulus of stiffness for paving mixtures incorporating each viscosity grade is 20-fold for the same low temperature conditions.

29. Insofar as the influence of the method for grading asphalt cements on low temperature transverse pavement cracking is concerned, since an increase in the low temperature modulus of stiffness of a pavement appears to result in a marked increase in low temperature transverse pavement cracking, Figures 3 and 6 indicate that grading asphalt cements by viscosity at 140 F would be far the worst method, since the range of modulus of stiffness would be 20-fold for each grade, for example AC 12 in Figure 3, while Figure 5 demonstrates that grading asphalt cements by penetration at 32 F would be the best method because the range of modulus of stiffness would be only 1.5-fold for each grade. On the other hand, with respect to high temperature construction operations, Figure 6 demonstrates that grading asphalt cements by viscosity at 140 F would be the best method since it would provide the narrowest range of viscosity at 140 F for each viscosity grade, while grading by penetration at 32 F would be the worst method because it would provide the widest range of viscosity at 140 F for each penetration at 32 F grade. Consequently, when both low temperature pavement performance and high temperature construction operations are considered, Figures 3 and 6 indicate that grading asphalt cements by penetration at 77 F provides the most desirable compromise. Figure 6 demonstrates that grading by penetration at 77 F provides a range of viscosity at 140 F intermediate between those of grading by viscosity at 140 F and by penetration at 32 F; and of much greater importance (Figs. 3 and 5), it provides a 4-fold range in pavement low temperature modulus of stiffness vs a 1.5-fold range for grading by penetration at 32 F and a 20-fold range for grading by viscosity at 140 F.

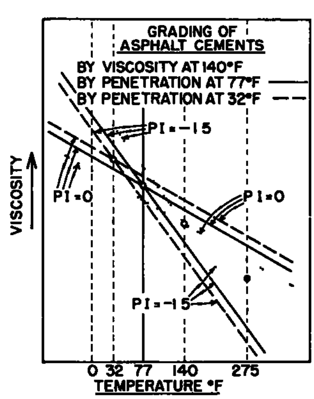

Figure 6. Consequences of grading asphalt cements by viscosity at 140 F, by penetration at 77 F, and by penetration at 32 F.

30. If restrictive specifications for asphalt cements are to be considered.

particularly when consideration is given to reducing or eliminating transverse pavement cracking at low temperatures, it is preferable that the restrictive specification should be based on grading asphalt cements by penetration at 77 F with a viscosity restriction, rather than grading them by viscosity at 140 F with a penetration at 77 F restriction. For example, 85/100 penetration asphalt with Line C in Figure 1 as a minimum viscosity requirement (PI = -1.0) is seen from Figure 3 to result in a permissible range in modulus of stiffness of about only 2.5-fold for slow loading (low temperature stress) at -10 F, 280,000 psi to 700,000 psi. On the other hand, it can be seen from Figures 1 and 3 that grading paving asphalts by viscosity at 140 F, for example 2000 ± 400 poises, with a specified range of penetration at 77 F of 60 to 120, results in a permissible range of pavement modulus of stiffness of about 6-fold (PI = 0.0 to PI = -1.2) for the same low temperature conditions, 200,000 psi to 1,200,000 psi. Since the wider the range of modulus of stiffness the more variable is the degree of low temperature transverse pavement cracking, these data imply that a restrictive specification based on grading by penetration and a minimum viscosity restriction, with its 2.5-fold range of modulus of stiffness, is greatly superior to a restrictive specification based on viscosity at 140 F and a wider permissible range of penetration at 77 F, with its 6-fold range of modulus of stiffness.

31. Figure 3 suggests that when it is guided by a need to greatly reduce or eliminate low temperature transverse pavement cracking, the selection of the penetration at 77 F grade of asphalt cement should be based on the maximum modulus of stiffness for the pavement that will just avoid low temperature pavement cracking throughout its service life. This will provide higher pavement stability for warm weather traffic. For example, when as illustrated by Figure 3, the lowest pavement temperature to be anticipated in some region is -10 F, if 85/100 penetration asphalt cement having a penetration index of 0.0 just avoids transverse cracking, then if an asphalt cement with a penetration index of -1.5 is being considered, its penetration grade should be 150/200. Figure 3 indicates that when all other factors are equal, paving mixtures containing either 85/100 penetration asphalt with a PI of 0.0, or 150/200 penetration asphalt with a PI of -1.5 will have the same modulus of stiffness at -10 F, and therefore, their low temperature service performance should be similar.

32. There appears to be a tendency in many areas to assume when selecting an asphalt cement, that only penetration grades with a high viscosity (high PI) should be specified. This fails to recognize the thoroughly demonstrated excellent service performance of many pavements that have been made with properly selected grades of low viscosity (low PI) asphalt cements. It also seems to be overlooked that low viscosity asphalt cements provide paving mixtures with less resistance to compaction by rolling; they provide a longer period of time during which compaction by rolling is effective, which is a very important advantage in colder climates; pavements compact faster to their ultimate density under traffic, which retards the rate of hardening of the asphalt binder in service, and lengthens pavement service life; and they provide pavements with substantially greater load carrying capacity per inch of thickness during the spring break-up period.

33. Finally, provided the old pavement is first covered with a substantial layer of stable granular material, there is some evidence that the use of softer asphalt cements in asphalt concrete overlays, will effectively reduce the amount of reflective cracking in the overlay that is presently occurring.

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