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Road Roughness and Slipperiness

Some Factors and Test Methods

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New Developments in BPR Roughness Indicator And Tests on California Pavements

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Since 1949 extensive use has been made in California of a road roughness indicator built according to plans furnished by the Bureau of Public Roads (BPR) with modifications developed at the University of California. A report of test results in California and of certain modifications in the design and calibration of the roughness indicator was presented at the 1951 annual meeting of the Highway Research Board. The roughness indicator was developed by the Bureau of Public Roads to provide standardizable equipment for measuring road surface roughness. Research with the California unit has continued during the past four years to improve its accuracy and consistency as a standardizable unit and to obtain records of road roughness of thousands of miles of pavements on the state highways, city streets and on bridges in all parts of California.

Tests were conducted to determine the limitations of accuracy provided by the BPR double ball clutch integrator design and also the accuracy of an integrator constructed with a commercially available clutch. Measurements were made to determine the effect of changing the size of the test tire from 6.00 by 16, which is no longer available, to a 6.70 by 15 size tire which is now available. The effects were measured of an improved leaf-spring bearing design and of the use of standard universal joints instead of ball-socket joints in attaching the dashpots to the frame of the trailer.

The effects of varying amounts of out-of-roundness of the test tire were measured on various road sections. It was found that the road roughness index in inches per mile for certain pavements was increased by approximately 50 percent due to an out-of-roundness of 0.05 inches which corresponds to the amount of out-of-roundness frequently observed in measurements of passenger car tires by attendants at tire and wheel alignment shops where tire truing work in done.

To protect the test tire and roughness equipment from damage and excessive wear in moving it from one test section to the next test section, a special outrigger trailer was developed to carry the test trailer in a suspended position by the use of a hoist and special clamping devices. Detailed shop drawings for the outrigger trailer have been prepared.

The results of roughness measurements for all of the major types of pavement surfaces used on state highways, city streets, and on various types of bridge floors are reported, analyzed and correlated with the design features, age of the pavement and construction methods used in building these surfaces.

● RESEARCH dealing with the measurement of road roughness using the Bureau of Public Roads (BPR) roughness indicator with modifications developed at the University of California, has been under way for the past seven years. This research is a part of a general study of road surface properties at the University of California which in addition to road roughness covers such items as skid resistance, road and tire noise, tire wear and tractive resistance. The preliminary phases of this study, including a description of the BPR roughness indicator and certain modifications of this equipment developed at the University of California, were described in papers presented at the Annual Meeting of the Highway Research Board in 1950 and 1951. These papers were published in the Highway Research Board Bulletins 27 and 37.

Measurements of road roughness have been made on many different types of pavements on the major state highways in the eleven state highway districts in California, and on many pavements in California cities and on the major bridges and freeway overpass structures in the San Francisco Bay Area. Repeated measurements were made on selected pavements to provide a record of seasonal and long-term changes in the roughness of these pavements. While a major objective in this study has been to assemble and analyze the roughness measurements on many different pavement types in all parts of California, an important objective has also been the development of testing equipment and of calibration and testing methods which will assure greater consistency and accuracy in the test results than were possible when work on this project was started.

The road roughness tests conducted over a seven year period have demonstrated that the basic design of the BPR roughness indicator is sound and that it provides the simplest and most accurate method to measure road roughness which has been developed to date. The modifications made at the University of California on the BPR design were intended to improve the accuracy and dependability of the equipment, and to make it a standardizable unit as the Bureau of Public Roads intended it to be. The addition of the direct recording oscillograph equipment has made it possible to obtain a graphical record which is very useful in analyzing the roughness data and in identifying locations for visual inspection to determine the type and probable causes of the roughness observed on the oscillograph records. Many tests were run to determine the effect on the roughness measurements due to actual wear or simulated wear of critical parts of the equipment, which may be expected after many years of operation of the equipment over many thousands of miles of road.

The results of the tests reported in this paper are intended to provide road roughness data and graphical records of road roughness for many different types of pavements under many different conditions under which the BPR roughness indicator may be used. In the discussion of the test data, an explanation will be offered for the variable results obtained under different test conditions with this equipment. In addition certain limitations will be pointed out concerning the use of the equipment which should be recognized in the interpretation and evaluation of the test data.

With passenger car speeds on many rural highways and on some urban expressways today averaging 50 to 60 mph and top speeds exceeding 70 mph, smooth pavements are necessary not only to provide a comfortable ride at these speeds but also to provide greater safety in steering and braking of cars driven at high speeds. Highway engineers in many states are recognizing the need for providing smooth pavements as is evident by the increased use of devices such as the BPR roughness indicator to measure road roughness. This equipment is now in use in about 15 states and on the basis of the many

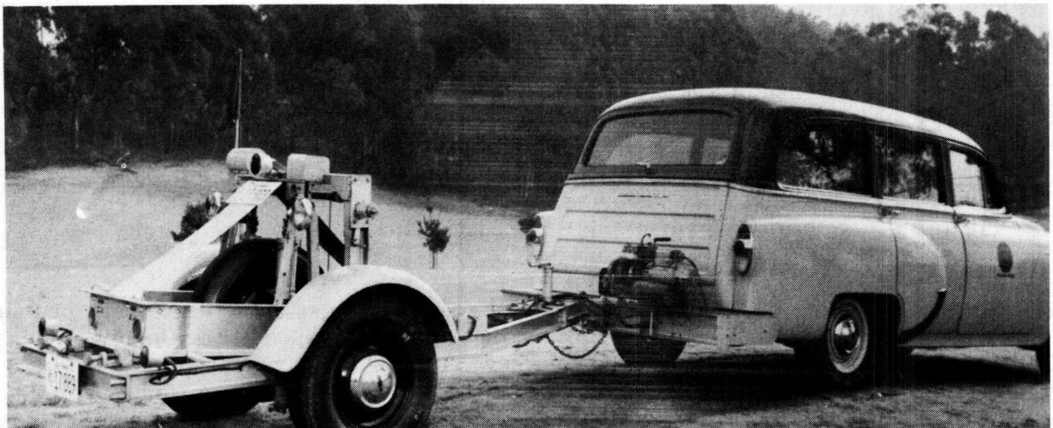

Figure 1. Road roughness indicator, outrigger trailer carrier and tow car.

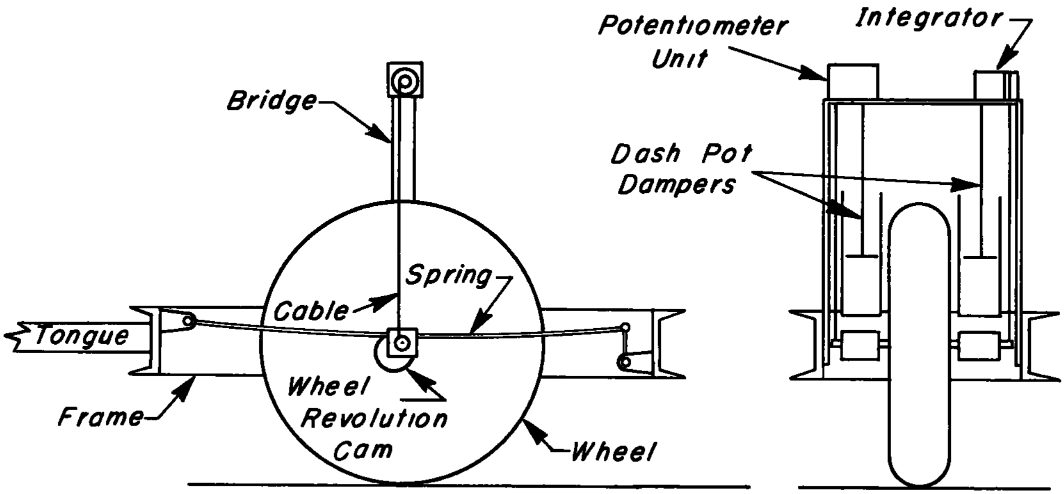

Figure 2. Schematic diagram of the essential elements of the Bureau of Public Roads road roughness indicator.

inquiries received during the past year concerning the equipment, it is expected that additional state highway departments will build similar equipment. The test results and operating experience with the BPR roughness indicator described in this paper should be of special interest to highway engineers who are now using similar equipment or who are contemplating using such equipment.

The roughness measurements can serve many useful purposes such as to provide a standard for new construction, for reconstruction and for maintenance. Roughness measurements are used by the Cities of Berkeley, Los Angeles and San Diego as a major item for rating the condition of streets and for programming street and highway work. The riding public judges a road largely by its smoothness or riding quality and it is, therefore, a matter of good public relations to construct and maintain pavements with as smooth a surface as is reasonably possible.

Description of the University of California BPR Roughness Indicator

The road roughness indicator used in the current research at the University of California was built in 1941 according to plans furnished by the U. S. Bureau of Public Roads. It was used in measuring the roughness of more than 1,000 miles of roads in Iowa, Kansas, Missouri and Wyoming in an extensive research program conducted by Iowa State College in cooperation with the Bureau of Public Roads. In 1949 this equipment was acquired by the University of California for the current research. Since 1949 it has been overhauled several times and modifications have been made for reasons mentioned above and to be described in greater detail in the discussion which follows.

A detailed description of the BPR roughness indicator is given in a paper by J. A. Buchanan and A. Catudal, entitled "Standardizable Equipment for Evaluating Road Surface Roughness," published in the 1940 Proceedings of the Highway Research Board. Briefly stated, this equipment consists of a single-wheeled trailer which is towed by a car or light truck (Figure 1). The BPR plans call for a standard four-ply 6.00 by 16 rib tread tire for the single wheel on the trailer. As the single-wheeled trailer is towed over a given section of road, the irregularities in the road surface transmitted through the tire to the axle of the wheel are measured in terms of the vertical movements of the axle. The vertical movements of the axle are transmitted by a wire cable to a double-acting ball clutch integrator which in turn transmits the accumulated vertical movements in inches to an electric counter mounted on a board in the tow car. A similar electric counter records the revolutions of the trailer wheel and thus provides an accurate measure of the travel distance. The roughness tests have been standardized at a speed of 20 mph and the measurements are recorded on a data sheet by an observer for each half mile section and/or at the end of each test section. The data are summarized by expressing the roughness of each section of road in terms of a standard unit known as the roughness index (RI), which is the roughness in inches per mile.

The essential elements of the BPR roughness indicator are shown in Figure 2. It should be noted that the wheel of the roughness trailer is supported by two light steel springs. Also, two specially designed dashpot dampers are attached to the axle of the wheel and the frame of the trailer to eliminate excessive bouncing or vibration of the tire as it rolls over rough spots on the pavement. Tests have shown that the tire and the dashpots provide excellent damping action such that the tire follows fairly closely the vertical profile of the pavement surface and thus the equipment provides a reasonably accurate measurement of the vertical movement of the wheel and tire on the paved surface.

Development of Direct Recording Oscillograph Equipment

The need for a graphical record of road roughness was discussed in both the 1950 and 1951 reports referred to above. Also, in these reports a description was given of the direct recording oscillograph equipment developed at the University of California and which has been used in the California tests to obtain graphical records of road roughness. This equipment has been very helpful for obtaining a detailed record of road roughness and for the analysis and interpretation of road roughness data as measured under many different conditions. In the latter part of this report many oscillograph records will be shown to aid interpreting the road roughness data and to provide an indication of the types of road roughness encountered under various test conditions.

Although wiring diagrams were shown in the 1951 report for the electronic amplifier, external bridge circuit and the power supply used with the amplifier, certain changes have been made in these wiring diagrams since 1951. The latest revisions of these wiring diagrams are shown in Figures 3 and 4.

IMPROVEMENTS IN THE DESIGN AND OPERATION OF THE BPR ROUGHNESS INDICATOR

The items in the design and operation of the BPR roughness indicator which were investigated at various times during the past seven years are the following:

1. Tire size, tread design, tire wear and roundness effects.

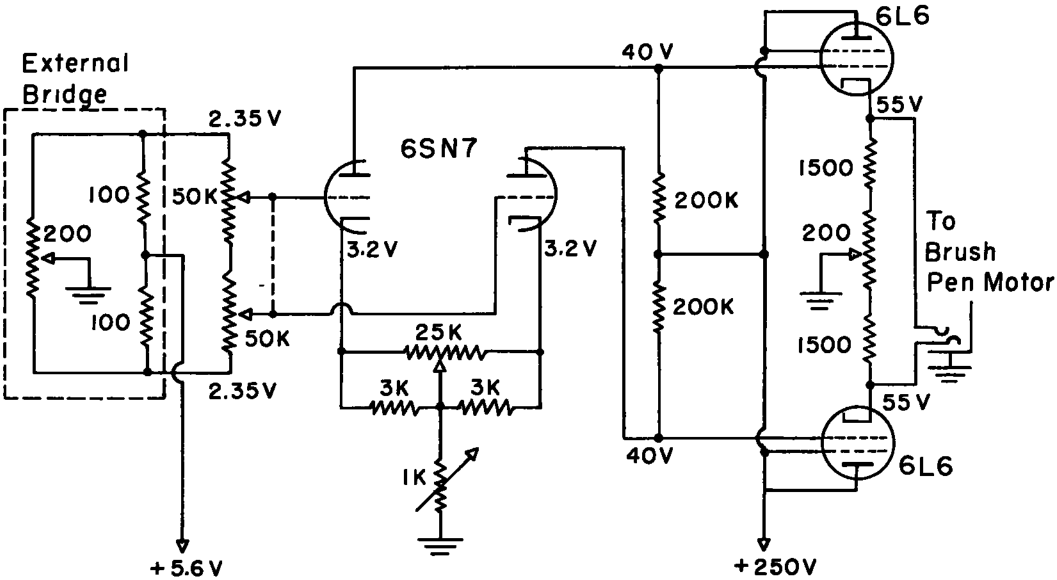

Figure 3. Electronic amplifier and external bridge wiring diagram used with BPR roughness indicator.

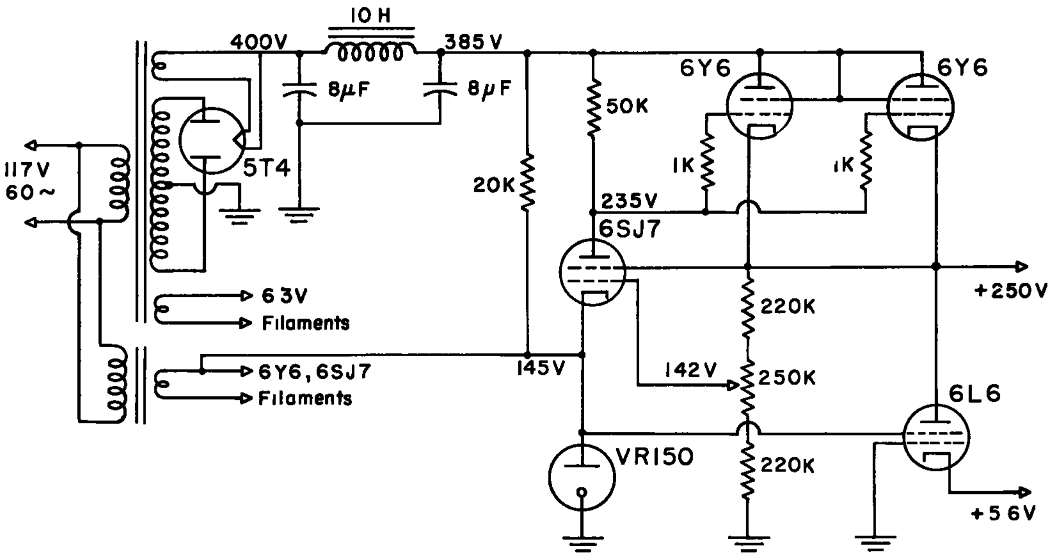

Figure 4. Wiring diagram for power supply used with electronic amplifier.

2. Oil leakage in the dashpot damping units, use of "0" rings, effect of worn joints and use of standard universal joints.
3. Improved leaf spring ball bearing design.
4. Improved integrator design - use of BPR clutch versus commercial clutch.
5. Development of equipment and method of calibrating integrator.
6. Improved wheel revolution counter design.
7. Effect of towing unit - use of outrigger trailer carrier.

Effect of Various Tire Factors

The early tests with a rib tread tire indicated that stone chips and gravel particles were picked up by the tread and were lodged in the grooves of the tread. This had the effect of increasing the roughness by an unpredictable amount. To correct this condition a four-ply 6.00 by 16 tire with a smooth tread was adopted for use on the University of California trailer.

In recent years the size of tires for popular priced cars has been changed from 6.00 by 16 to 6.70 by 15. Accordingly the U. S. Rubber Company, which manufactured smooth 6.00 by 16 tires for use on the BPR roughness indicator, discontinued making the 6.00 by 16 tire several years ago. This company is now, however, making a 6.70 by 15 smooth tread tire for use with this equipment. This new tire may be obtained on special order from the factory.

The University of California roughness trailer was modified to permit running tests with the older size 6.00 by 16 tire and, also, with the new 6.70 by 15 tire. Tests were run with both of these tires on six different road surfaces with varying amounts of roughness. The results of these tests, shown in Table 1, clearly indicate that the change in tire size produced for all practical purposes no change in the roughness indexes as measured on these six surfaces. For four of the surfaces the difference in the roughness index was only 1 in. per mile which is well within the experimental error. On the roughest pavement, the difference amounted to 4 in. per mile but here again this is within the experimental error. The use of the new tire introduced a change in wheel revolutions per mile as

TABLE 1

COMPARISON OF TEST TIRES

Test Section	Pavement Type	Roughness Index, in per mile: 6 00-16 Synthetic Rubber Tire	6 70-15 Synthetic Rubber Tire
1	Bituminous	36	37
2	P C Concrete	60	61
3	Bituminous	73	75
4	Bituminous	133	132
5	P. C Concrete	157	158
6	Bituminous	295	291

was expected but even this change was rather small. For the new 6.70 by 15 tire, the wheel revolutions per mile under standard test conditions average 742 as compared to 736 for the older 6.00 by 16 tire.

TABLE 2

COMPARISON OF TEST TIRES

		Roughness Index, in. per mile	
Test Section	Pavement Type	Smooth Tread Natural Rubber Tire	Smooth Tread Synthetic Rubber Tire
1	Bituminous	68	66
2	P. C. Concrete	72	72
3	P. C. Concrete	166	164

Tests were run to compare the roughness index for three road surfaces using a 6.00 by 16 synthetic rubber tire which was less than one year old and a 6.00 by 16 natural rubber tire 10 years old. The results of these tests given in Table 2 show that there was no measurable effect due to age or to the type of rubber used. It should be mentioned here, however, that in making these comparison tests both tires were carefully checked for roundness prior to the test. As will be shown in the discussion which follows, large errors in the road roughness measurements are obtained if the tests are run with tires which are more than 0.03 in. out of round.

Effect of Tire Out-of-Roundness

In the early stages of this study, it was noted that if the test tire was not perfectly round, an increase in the roughness index was obtained which was directly related to the out-of-roundness of the tire. Since the effect of out-of-roundness of the test tire did not appear to follow a fixed pattern, it was decided to make a special study of this tire factor.

In general, tire out-of-roundness is of two types. One of these is in the form of a flat spot on the tire due to locked-wheel braking or for synthetic rubber tires, the flat spot may be caused by keeping the tire in a loaded position for a sufficient length of time to cause plastic deformation of the rubber in the tread. Another form of tire out-of-roundness is due to the wheel or tire or both not being centered accurately on the hub or axle of the wheel assembly.

The method used in this study to obtain an accurate measurement of tire out-of-roundness was by the use of a Federal Dial indicator with the tire and the indicator

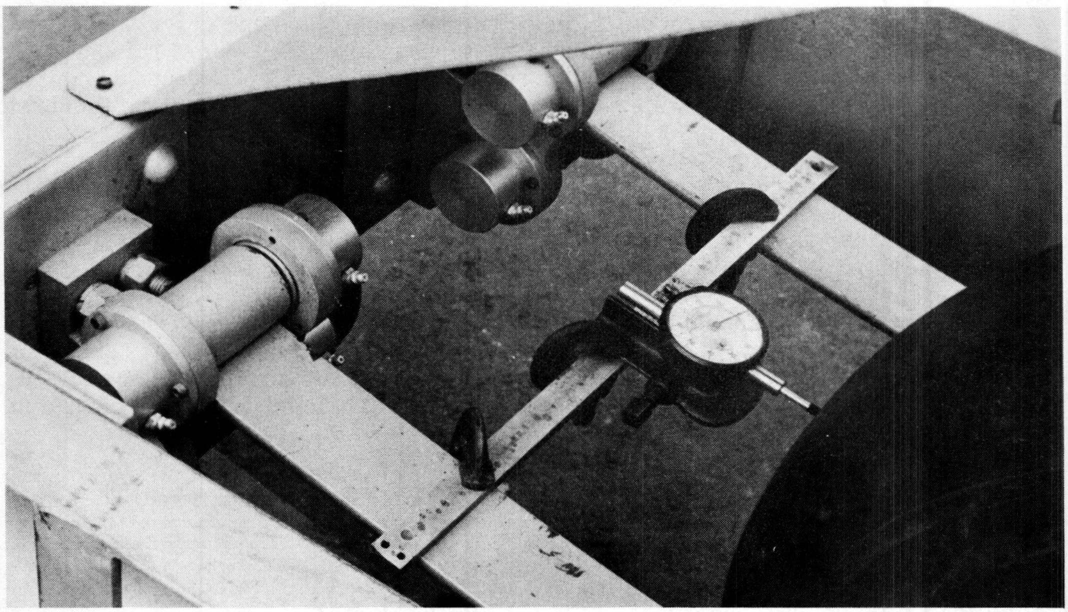

Figure 5. Method of measuring tire out-of-round using Federal Dial indicator. Also shown are the special grease cups with grease fittings used in redesign of ball bearing mountings for leaf springs.

mounted on the trailer as shown in Figure 5. With this method, readings could be made to the nearest 0.001 inch on the indicator.

To determine the effect on the road roughness measurements of varying amounts of out-of-roundness of the tire, the wheel was mounted in four different off-center positions resulting in out-of-roundness varying from 0.017 in. to 0.100 in. Tests were

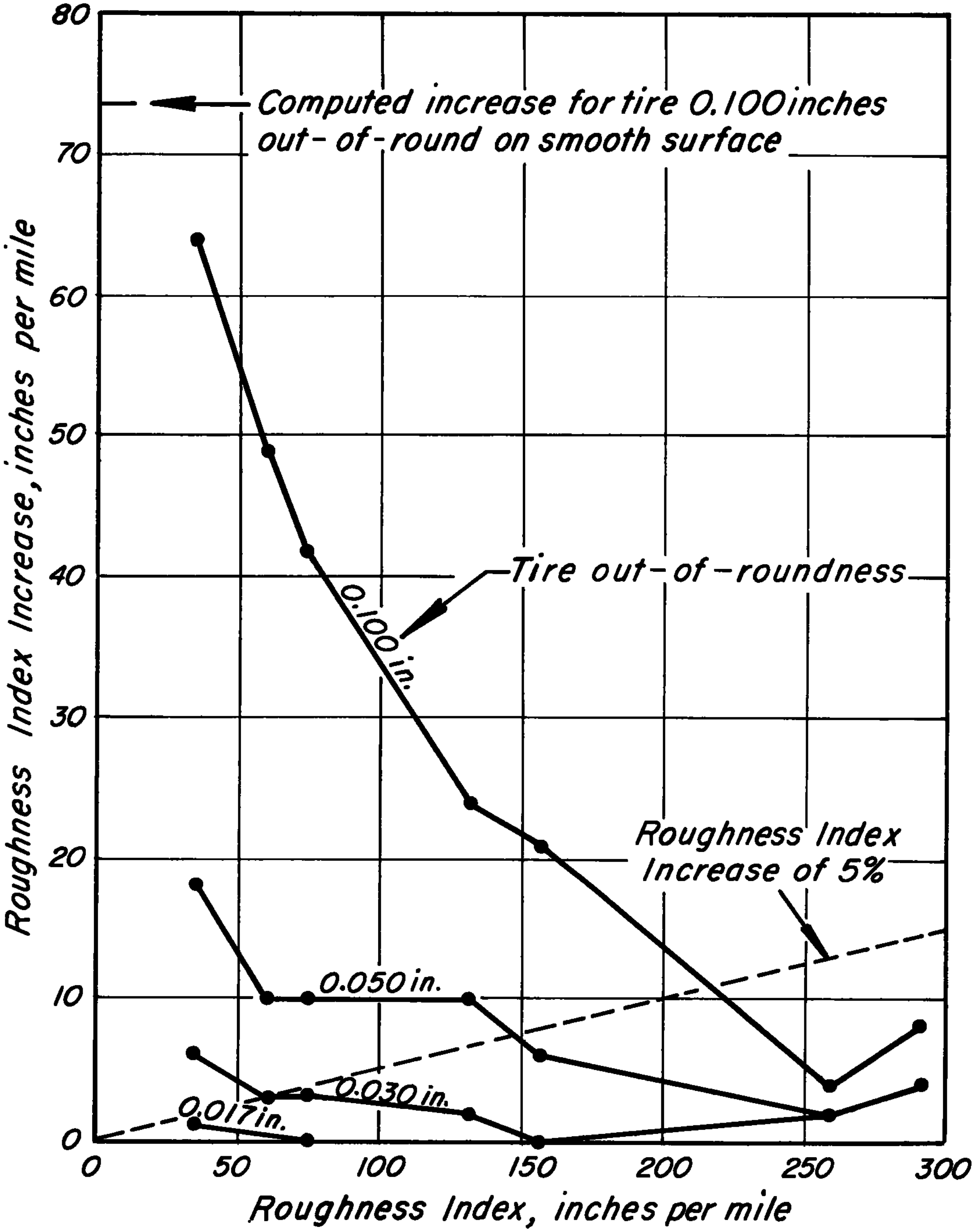

Figure 6. Effect of tire out-of-roundness on the roughness index measurement on various road surfaces.

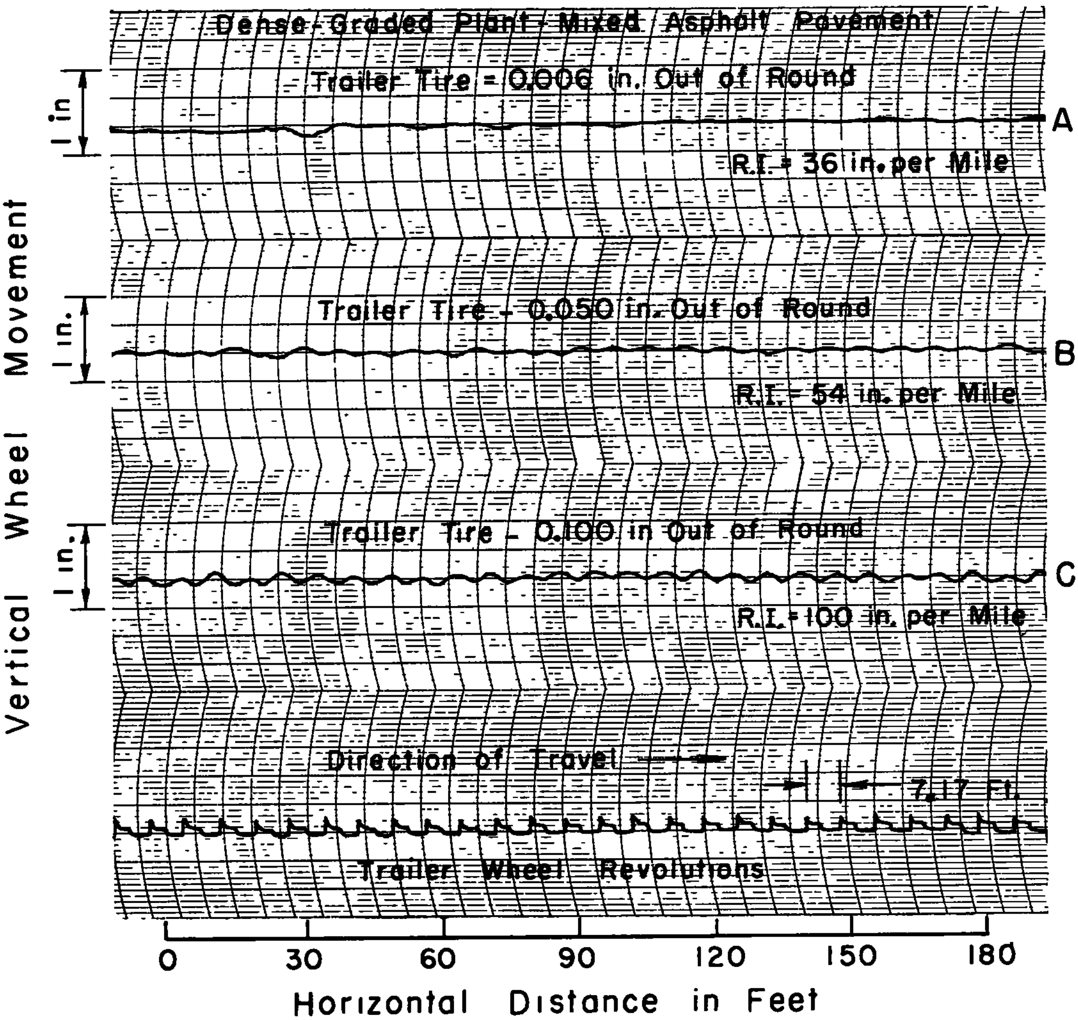

Figure 7. Roughness oscillograph records showing the effect of the use of a smooth trailer tire with varying amounts of out-of-roundness in tests on the same pavement.

then run on seven different road surfaces with roughness indexes for the standard test conditions ranging from 35 in. per mile to 300 in. per mile. The results of these tests, shown in Figure 6, indicated that the maximum increase in the roughness index due to tire out-of-roundness was obtained on the smoothest pavement. On this pavement with a tire out-of-roundness of 0.100 in., the increase amounted to 64 in. per mile which is approximately a 200 percent increase. By assuming that the full 0.100 in. out-of-roundness is effective during each wheel revolution the computed increase in road roughness will amount to 74 in. per mile which is only 10 in. per mile greater than the measured increase on a smooth pavement.

The tests with the off-centered tire indicated that the increase in road roughness falls off sharply as the pavement roughness is increased. Thus, with a pavement roughness of 250 in. per mile, the effect of tire out-of-roundness was almost entirely eliminated since on this pavement the increase for the 0.100 in. out-of-round tire amounted to less than 5 in. per mile.

The effects of varying amounts of tire out-of-roundness on the same smooth asphalt pavement are shown in graphical form in the oscillograph records (Figure 7). These records provide some interesting patterns. It should be noted in Figure 7 that with the test tire 0.100 in. out-of-round, a sine-curve roughness pattern of considerable amplitude is obtained on the oscillograph record. The roughness of a smooth asphalt pavement will be raised from 35 in. per mile to 100 in. per mile with the test tire 0.100 in. out-of-round, and thus make this pavement appear to be rough riding when actually the rough ride should be attributed to an out-of-round tire condition.

While no attempt was made to determine the extent of tire out-of-roundness of tires on passenger cars in service, the reports of service stations equipped to do tire truing work, indicated that tire out-of-roundness of 0.05 in. to 0.10 in. was quite common. A common cause of tire out-of-roundness is that which results from the flat spots due to locked-wheel braking. To determine the extent of the flat spot tire wear caused by locked-wheel braking, braking tests were conducted and the tire wear resulting from these tests was measured in $\frac{1}{1000}$ in. units. The results of these tests are shown in Table 3. It should be noted that the wear in the central portion of the tread is only about one-half the wear along the outer edge of the tread. However, in a single stop from 60 mph with a skidmark 150 ft long, the depth of tread removed from the tire amounted to 0.070 in. This is evidence that locked-wheel braking can cause an out-of-round tire condition which can be as much of a factor in causing a rough ride as a poorly constructed pavement.

It is evident from the above discussion that special precautions should be taken to keep the BPR roughness indicator test tire in an in-round condition. Tire truing machines are now being used by certain tire service station operators which can reduce tire out-of-roundness to ± 0.001 in. with the tire mounted on the machine and with no load on the tire. It has been found that the mounting studs on the wheel hub may be off-center by as much as 0.010 in. and thus the tire may be out-of-round when mounted on the trailer due to an off-center mounting. To correct this error the studs should be carefully centered and a final check of tire-roundness made as shown in Figure 5. For accurate roughness measurements, the maximum permissable variation in test tire out-of-roundness as measured with a precision dial indicator should be ± 0.010 in. and the preferred maximum variation should be ± 0.005 in.

It should also be noted that synthetic rubber tires develop flat spots when standing in the same spot supporting a load. For this reason, the test tire should be held in an unloaded position except during tests as an extra precaution to keep the test tire within the permissable amount of out-of-roundness.

TABLE 3

PASSENGER CAR TIRE WEAR IN ONE SPOT OF TREAD OBTAINED IN A LOCKED-WHEEL BRAKING TEST

Initial Speed, mph	Average length of skidmarks, feet	Tire tread wear, $\frac{1}{1000}$ in. Along outer edge of tire tread	In central portion of tire tread
20	17	8	4
30	33	15	8
40	62	29	15
50	105	49	26
60	150	70	37

TABLE 4

EFFECT OF "0" RING INSTALLATIONS CALIFORNIA TESTS

Test Section	Pavement Type	Roughness in in. per mile for various types of "0" ring installations[a] 1	2	3	4
1	Bituminous	68	67	57	65
2	Bituminous	140	140	120	140
3	Concrete	166	170	155	168

[a] "0" ring installation:
1 "0" rings installed with recommended depth of cut in bushings of 0 090 in 6,000 miles of highway travel, very little fluid leakage.
2 Without "0" rings.
3. New "0" rings installed with recommended depth of cut in bushings of 0 090 in.
4 New "0" rings installed with depth of cut in bushings increased until very little drag was felt on piston rod and still with no fluid leakage

TABLE 5

EFFECT OF "0" RING INSTALLATION U S. BUREAU OF PUBLIC ROADS TESTS

Test Section	Pavement Type	Road Roughness (in per mile) Without "0" rings	With "0" rings
1	Concrete	142	119
2	Concrete	126	90
3	Bituminous	122	86

Effect of Dashpot Damping Units on Road Roughness

Observations in tests with the University of California roughness indicator and with a similar unit built by the City of San Diego, indicated that there were three or four features in the design and operation of the dashpot damping units which, if not properly controlled, could introduce large variations in the roughness measurements.

Tests by the Bureau of Public Roads indicated that the viscosity of the oil used in the dashpots must be standardized and carefully controlled to obtain consistent results. Likewise, the height of the oil level was found to be an important item. Oil leakage presented quite a problem in some of the tests in California and to correct this difficulty "O" rings were installed in the bushings of the dashpots. It was found, however, that if the "O" rings were installed with a tight fit, considerable drag was introduced on the piston rod as it was raised and lowered. The effect of this drag resulted in a reduction of the roughness index values as shown in Tables 4 and 5. The tests with the University of California trailer indicated reductions of 10 to 15 percent in the road roughness values due to tightly fitting "O" rings. Similar tests by the Bureau of Public Roads indicated reductions as high as 30 percent according to the data shown in Table 5.

The data in Table 4 show that if the "O" rings are installed with a depth of cut in the bushings until very little drag is felt when raising and lowering the piston, no error is introduced due to the use of "O" rings and oil leakage can still be held to a minimum.

In this connection it should be mentioned that much of the wear in the dashpot bushings probably resulted from hauling the test trailer from one test site to another at speeds considerably in excess of 20 mph over fairly rough roads. Several years ago a special outrigger trailer carrier was built at the University of California to haul the test trailer from one test site to the next. The test trailer has for the past two years been operated only at 20 mph on the test sections and the difficulties with excessive bushing wear and oil leakage have for the most part been eliminated.

TABLE 6
COMPARISON OF JOINTS IN TEST UNIT AND EFFECT OF WEAR OR END-PLAY FOR BALL-THRUST JOINTS

		Roughness Index, in per mile			
			BPR Ball-Thrust Joints		
Test Section	Pavement Type	Standard Universal Joints	No End-Play	1/16-in end play, each side	1/8-in end play, each side
1	Bituminous	68	70	94	104
2	Bituminous	113	112	144	180
3	Bituminous	174	176	202	240

Another feature in the design of the dashpot units which our tests demonstrated could be responsible for large errors in the roughness measurements, were the ball-thrust joints where, after many thousands of miles of operation, excessive wear at the joints caused a small amount of end-play. The magnitude of the error in the road roughness values caused by varying amounts of wear and end-play at the ball-thrust joints is indicated in the test results given in Table 6 for three different pavements. To eliminate the end play referred to above, the ball-thrust joints were replaced with standard universal joints. Test results are given in Table 6 which show that the roughness index remained the same for tests on the same pavements for the trailer equipped with universal joints as for the trailer with the BPR ball-thrust joints with no end-play. With 1/8 in. end-play at the ball-thrust joints, the roughness index values were increased approximately 50 percent, as for example, from 70 in. per mile to 104 in. per mile and from 176 in. per mile to 240 in. per mile. A 50 percent error is much too large for satisfactory operation of road roughness equipment and it is evident that either the ball-thrust joint design should be changed or special precautions taken to eliminate end-play at the ball-thrust joints by providing a finer adjustment for seating the ball in the socket.

Improved Leaf Spring Ball-Bearing Design

The suspension system for the roughness trailer was designed to be as nearly frictionless as possible to prevent the variable damping effects commonly observed when an assembly of leaf springs or certain other types of suspension systems are used. For this reason light single leaf springs and ball bearing mountings were used. In general, this design has been satisfactory except for the difficulty in keeping the ball bearings clean and well lubricated. Even with sealed bearings, it was found that water and dirt accumulated in the bearings, caused corrosion and pitting of the bearings and prevented the desired low friction action of these bearings. Accordingly special grease cups with grease fittings were designed for all of the ball bearing mountings. Drawings for the special grease cup design and the mountings may be obtained through the Highway Research Board. In the new design the shields were removed from the ball bearings, thereby facilitating flushing the bearings with grease through the grease fittings by the use of a grease gun. The operation of flushing the ball bearings with grease requires very little time and we recommend that it be done at least once or twice a week if the roughness trailer is in continuous use.

Integrator Design - BPR Clutch Versus Commercial Clutch

The integrator is the most important part of the BPR roughness measuring mechanism. It consists of an over-running double ball clutch which accumulates or integrates the vertical movement in one direction only of the axle on which the test wheel is mounted. Over the seven year period in which this equipment has been used in California, there have been a number of difficulties encountered which contributed to inaccurate readings with the BPR integrator design. Some of these factors were discussed in the 1951 report, including changes in the design of the integrator. Additional improvements have been made in the integrator since 1951 and tests have been conducted to indicate the limitations of accuracy of the integrator with various modifications developed at the University of California.

Such factors as dirt and dust in the case, corrosion of the metal surfaces in the integrator, misalignment of the main shaft in the integrator, stretch in the cable, arching between the carbon brush and commutator, and variable tension in the springs of the rear ball clutch were items which caused most of the trouble with the integrator. The calibration device described in the 1951 report indicated that for certain conditions described above the integrator would "grab" or develop slippage which resulted in lower values of road roughness than the true values and for some of the other conditions the integrator would overthrow or introduce extra counts which resulted in higher readings than the true values.

The use of the six-pronged cam and the micro precision switch described in the 1951 report, eliminated the errors caused by the brush-commutator design. Since then a further modification has been made in the micro precision switch by changing it from a type L2 to a type W22 (Figure 8). The latter type switch does not improve accuracy but it provides a more compact design of the integrator.

Since erratic results have been obtained on certain occasions with the BPR double ball clutch design, a special study was made to determine the reasons for the erratic results and to find out how the errors caused by the clutch could be reduced or eliminated. To eliminate or reduce the errors caused by dust and water entering the integrator, a new dust-proof case was built, with heavy felt covering the cable opening through which the steel cable has to operate. While it was observed that the heavy felt was cut by the cable leaving a larger cable opening than desired, there was no indication that an objectionable amount of dust entered the case at this point. To prevent the formation of rust on the inner and outer races of the ball clutch, it was decided to experiment with a hard chrome finish for these parts. It was found, however, that with the hard chrome finish, all wedging action of the steel balls in the races was lost, causing 100 percent slippage and thus the clutch did not function at all. A new clutch was then built to BPR specifications and installed in the integrator. It was oiled using the light mineral oil specified by the BPR. The small clutch ball springs were brought to uniform tension to hold the steel balls firmly in place. The entire integrator was built with very close

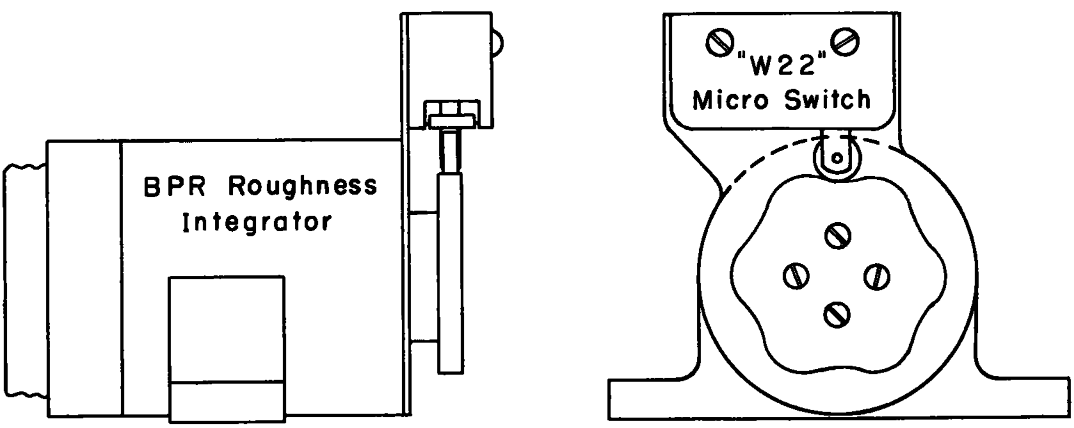

Figure 8. Modification to the BPR roughness integrator.

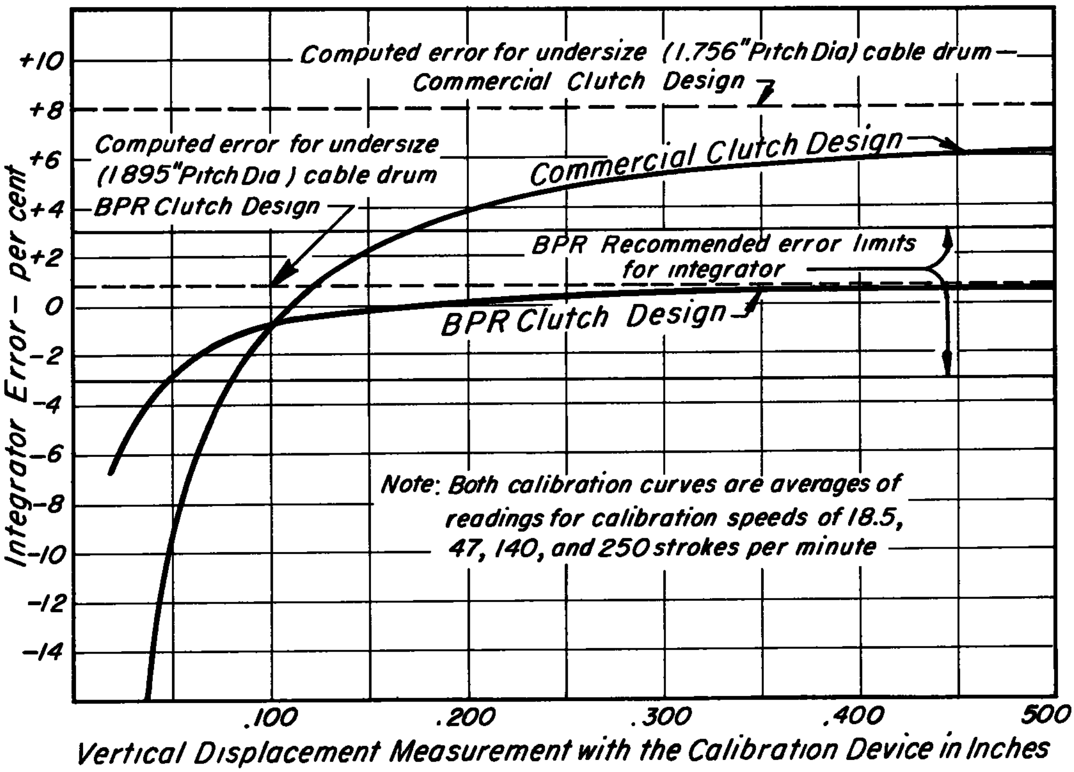

Figure 9. Comparison of the integrator error obtained in the calibration of the commercial clutch integrator and the BPR clutch integrator.

tolerances as called for by the BPR plans and specifications. The integrator was then calibrated at speeds ranging from 18½ to 250 strokes per minute.

The calibration results for the BPR clutch design shown in Figure 9, indicate that for vertical displacements from 0. 10 in. to 0. 50 in., the error in the measurements was within ± 1 percent. For vertical displacements under 0. 10 in., the error increased to about -6 percent for a displacement of 0. 025 in. These calibrations show that a mechanical clutch such as the BPR over-running double ball clutch will not pick up displacements under 0.05 in. within the ± 3 percent allowable error recommended by the Bureau. There are two design features of the BPR integrator which provide an explanation for the limitations in the sensitivity of the integrator. They are (1) the stretch in the steel cable which was discussed in the 1951 report and (2) the limit in displacement obtained as the result of the wedging or ratchet action and a certain amount of slippage of the steel balls in the clutch. The calibration curves in Figure 9 and the analysis and observations of the operation of the BPR integrator clearly indicate that at some point near a displacement of 0. 010 in., the limit of sensitivity of the mechanical type clutch is reached and that displacements lower than about 0. 005 in. are not recorded by this instrument. It should be recognized, however, that a displacement of 0. 005 in. per wheel revolution provides a maximum computed roughness index of 4 in. per mile which is so small that it has no practical significance in the evaluation of pavement roughness even for the smoother pavements where the measured roughness index values are in the range of 40 to 60 in. per mile.

Consideration was given to the possibility of reducing the cost of the integrator and possibly of improving the clutch action by using a commercial clutch. Accordingly, two Morse cam clutches No. B-203 were purchased at a cost of only $10. 00 each. A new integrator was built incorporating the commercial clutches in the design of the integrator. The Morse cam clutch was described by the manufacturers as a self-contained ratchet with an infinite number of teeth - one directional drive-over-running and free-wheeling.

The calibration results for the integrator built with the commercial clutch are given in Figure 9. It is evident from the calibration data that the commercial clutch provides the desired accuracy with ± 3 percent over such a small range of displacements (from 0. 08 in. to 0. 175 in.) that it is questionable if a Morse cam type clutch should be used in the design of the integrator. The BPR ball clutch design provided far superior accuracy on the basis of the calibration data in Figure 9 and is therefore recommended as the preferred design for a mechanical type clutch.

Test runs were made on many different pavements using the BPR roughness indicator equipped with both the BPR clutch and the commercial clutch integrator. In these tests both integrators were in operation at the same time. The results of the tests are shown in Figure 10. While the variations in units per mile do not appear to be large,

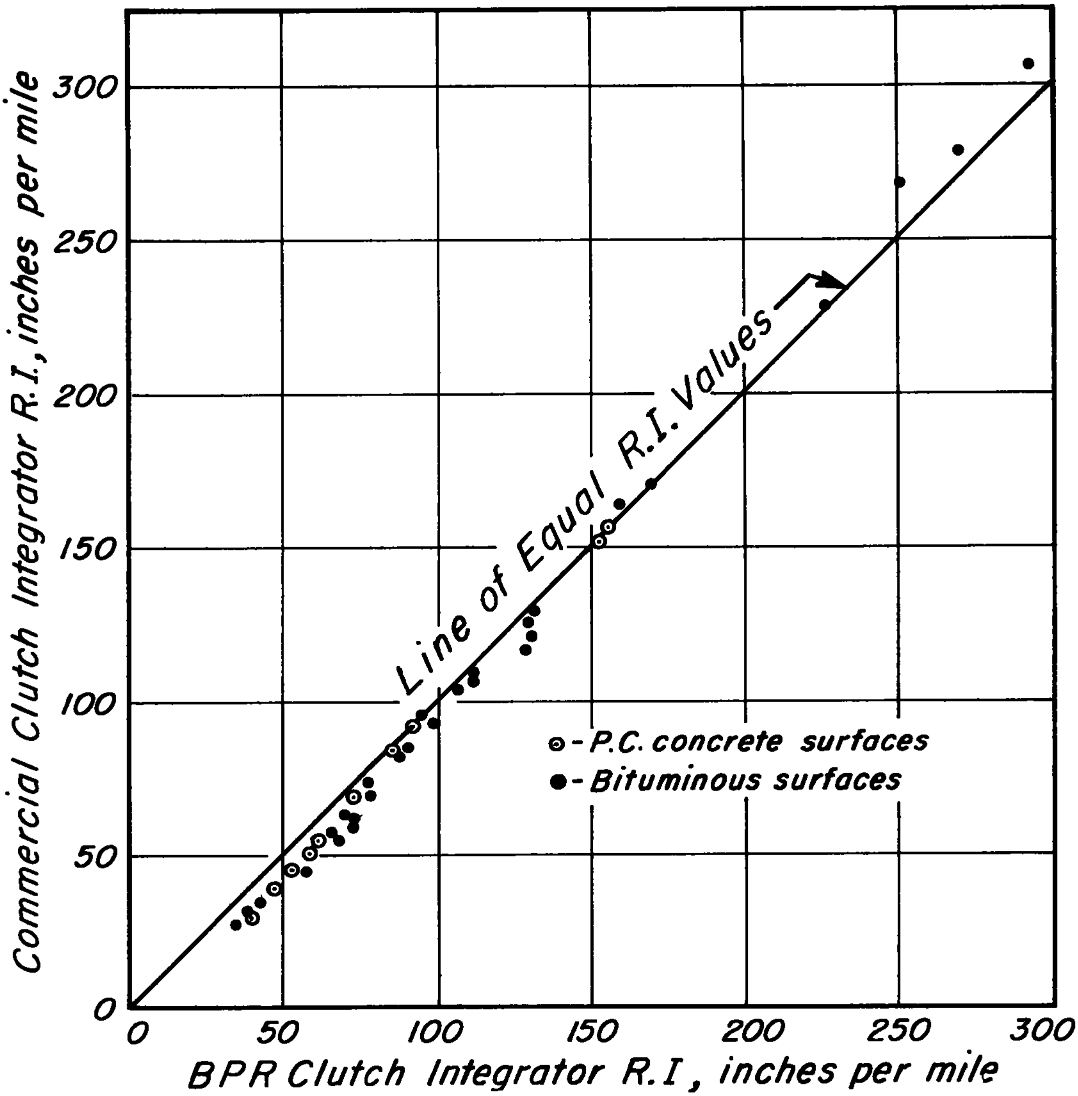

Figure 10. Comparison of roughness index (R.I.) on various surfaces measured with commercial clutch integrator and with BPR clutch integrator.

the errors or variations on a percentage basis are of the order of magnitude of ± 10 percent to ± 20 percent and thus do not comply with the ± 3 percent error specified by the Bureau. The results of these tests support the above recommendation that the Morse cam commercial clutch should not be used.

Equipment and Method for Calibrating Integrator

It is evident from the above discussion that the integrator is a rather complex and sensitive measuring device which must be ruggedly built to prevent damage when operated over rough pavements. It is highly desirable to have a fast and convenient method of calibrating the integrator periodically in the laboratory without requiring the operation of the tow car and the entire roughness indicator. The calibration unit developed by the University of California, described in the 1951 paper, has provided excellent calibration results during the past six years.

The data shown in Figure 9 were obtained by the use of the calibration device and it is doubtful if they could have been obtained by any other method. Since the integrator is a measuring device, it is highly desirable to know the limitations of the device or the errors which may be expected in its use. Also, for routine tests, it is desirable to be able to make use of a fast and convenient method of checking the integrator to be sure that it is functioning properly. All of this is accomplished with the University of California integrator calibration device.

Improved Wheel Revolution Counter Device

To obtain the desired accuracy in the measurement of road roughness, it is necessary to have an accurate measurement of the distance traveled by the roughness indicator. In the BPR design of the roughness indicator, distance is measured by obtaining a record of the number of wheel revolutions of the test wheel on the roughness trailer. A contact switch operated by a cam on the hub of the test wheel closes the circuit of the magnetic counter once for each wheel revolution. With 736 wheel revolutions per mile for the 6.00 by 16 tire, it is evident that for satisfactory results when operating the roughness trailer over thousands of miles of road, the contact switch must be well built to keep out dust and water which are certain to cause excessive wear of the contact surfaces and fouling of the contact point. The sliding surfaces on the cam and plunger should be made of hardened steel and provision should be made in the design of this part of the contactor mechanism to keep the cam wiped clean and the surface lubricated with a light oil by the use of a felt wick and oil cup attachment which is a design feature not shown on the BPR plans.

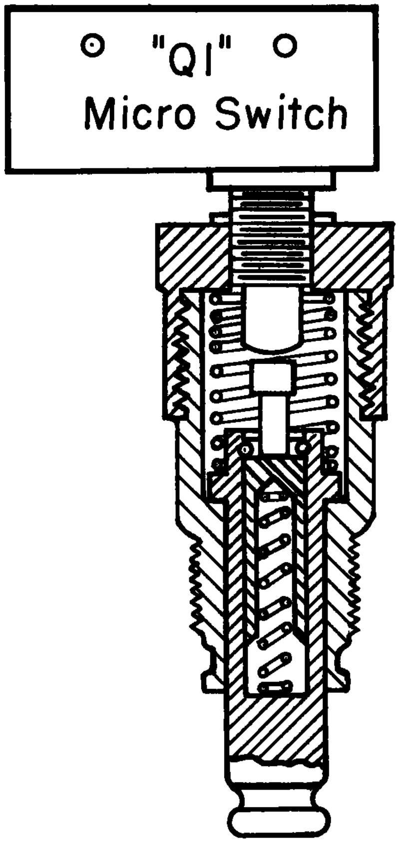

Figure 11. Use of micro switch in revolution counter design for BPR roughness indicator.

The contact switch when built as shown on the BPR plans was continually giving erratic results due to wear, water and dirt fouling the contact point. The design of the contact switch was modified as shown in Figure 11 to make use of a sealed-in micro switch, Type Q-1. The installation of the felt wiper and oiler and the micro switch eliminated all of the difficulties previously encountered with the revolution counter mechanism. The new design has given excellent service during the past two years. Of course, the plunger of the contacter requires cleaning about once a month or every 1,000 miles or as needed depending upon its exposure to dust, dirt and water.

TABLE 7

ROAD ROUGHNESS INDEXES MEASURED IN REPEATED TESTS ON SELECTED CALIFORNIA PAVEMENTS

U S. 40 - Fairfield Bypass, P. C. Concrete

Test Date	Roughness Index, in per mile: No. Joints Continuous Reinforcement	Standard 15 foot joint spacing
4-18-50	38	42
8- 2-50	41	42
10-11-50	39	40
4-18-51	44	48
9-21-51	40	44
8- 5-52	45	45
9-11-52	52	61
7-28-53	45	45
5-25-54	48	50
10-24-55	50	40

Eastshore Freeway, Fallon St to 23rd Ave
Four Lanes of P. C Concrete - Passing Lanes

Test Date	Roughness Index, in. per mile: Southbound	Northbound
7-21-49	53	56
1-24-50	55	54
2- 8-50	48	50
4-12-50	50	55
7-26-50	51	47
9-26-50	73	68
9-20-51	82	72
9-10-52	82	72
7-27-53	73	70
10-14-55	68	68

Oxford St, City of Berkeley, P C Concrete

Test Date	Roughness Index, in per mile
8-16-49	158
9-20-51	164
12-11-51	168
9-10-52	168
7-27-53	158
4-15-54	165
10-20-55	160

Eastshore Freeway, Plant-Mixes Surface, Open-Graded

Test Date	Roughness Index, in per mile
9-19-51	68
12-11-51	70
9-10-52	72
7-27-53	72
4-15-54	74
10-13-55	72

U.S 40, Vicinity Vacaville, Seal Coat, ¾ in. Aggregate

Test Date	Roughness Index, in per mile
8- 2-50	73
4-18-51	83
9-21-51	77
9-11-52	69
12- 8-54	66

Calibration of Towing Unit-Use of Outrigger Trailer Carrier

While the BPR roughness indicator is a fairly simple and ruggedly built piece of equipment, it is a device which for satisfactory results should measure vertical displacements in highway pavements at 20 mph with a high degree of accuracy and with a precision of the order of 5 to 10 thousandths of an inch. Calibration of the integrator is an important aid in checking the accuracy of the equipment but there are many other parts of the roughness indicator such as the dashpot assembly, the revolution counter, the test tire and the ball bearing spring mountings which need periodic checking.

The method used for checking the accuracy of the University of California roughness indicator as a complete unit, has been by running repeated tests over a selected section of concrete pavement built to a high standard in a location where surface and structural failures of the pavement are not likely to develop. The continuously reinforced concrete pavement on U. S. 40 on the Fairfield Bypass has proven to be an excellent pavement for use in checking the general performance and accuracy of the entire roughness unit. Repeated tests have also been run on sections of the Eastshore Freeway, a city street in Berkeley and on an asphalt pavement with a seal coat on U. S. 40 near Vacaville.

The results of repeated tests are shown in Table 7. These tests were started in 1949 and have been run each year on certain pavements up to and including 1955. It should be noted that the roughness index values measured in September, 1952, on certain pavements were higher than usual. A careful checking of the equipment indicated that the test tire used in these tests had developed a flat spot making it more than 0.010 in. out-of-round. The increased roughness values measured on the Eastshore Freeway since 1950 were due largely to settlement and structural failure on a short section of this pavement. This section of pavement has been resurfaced and the roughness index values were reduced somewhat but the average values were still higher in the 1955 tests than in the 1949 tests.

Comparison of Roughness Indicator Test Units

At the present time, three BPR roughness indicator units are in operation on the

TABLE 8

COMPARISON OF TEST UNITS OREGON HIGHWAY DEPARTMENT AND UNIVERSITY OF CALIFORNIA ROUGHNESS TEST UNITS, NOVEMBER 14, 1951

Test Section	Roughness Index, in per mile State of Oregon Roughness Trailer	Univ of California Roughness Trailer
Oregon Redwood Highway		
Mile 1	82	92
Mile 2	84	94
Crater Lake Highway		
Mile 1	79	90
Mile 2	92	98

TABLE 9

COMPARISON OF TEST UNITS CITY OF SAN DIEGO AND UNIVERSITY OF CALIFORNIA ROUGHNESS TEST UNITS APRIL 28, 1954

Test Section	Roughness Index, in per mile City of San Diego Roughness Trailer	University of California Roughness Trailer
Florida Drive	89	94
Florida Street	214	219
Madison Ave	199	205
Arthur Ave	446	459
Marlborough Dr	246	248

West Coast. In addition to the University of California unit, the Oregon State Highway Department built and is running tests with a unit and the City of San Diego also has a unit. Tests were run in November, 1951 on the same sections of pavement to make a comparison of the roughness measurements obtained with the Oregon unit and the University of California unit. The results of these tests are shown in Table 8. Similar tests were run in April, 1954 to compare the roughness measurements obtained with the City of San Diego unit and the University of California unit. The results of these tests are shown in Table 9.

In general, the roughness index values obrained in the comparison tests of the Oregon, University of California and the San Diego roughness indicator units were of the same order of magnitude although the values for the University of California unit were 5 to 10 percent higher than for the other two units. In checking the various parts of the three units, it was concluded that there was less friction or damping action in the dashpots and leaf spring ball bearings of the University of California unit than in the other two units.

It is interesting to note here that the City of San Diego experienced some difficulty during the first month of operation of their roughness unit in obtaining consistent results. Considerable extra damping effect was observed when running tests on the same

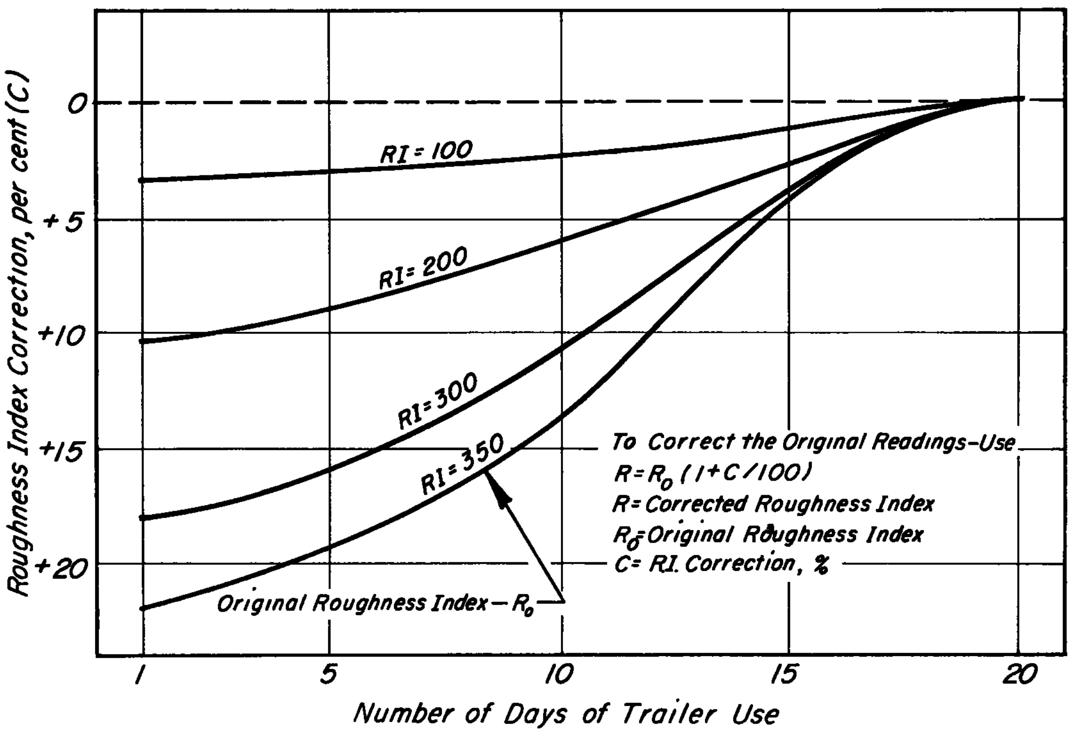

Figure 12. Correction curves used by the city of San Diego for adjusting the original roughness index readings during break-in period of BPR roughness indicator.

pavement. This extra damping effect decreased gradually during the first month of operation of the unit which was referred to as a "break-in" period. Correction curves were prepared as shown in Figure 12 for use in making corrections of the roughness index of pavements on which tests were run during the "break-in" period.

TABLE 10

EFFECT OF TOWING UNIT ON ROUGHNESS INDEX MEASUREMENTS ON P C CONCRETE PAVEMENT, U S 40, FAIRFIELD

Test Section	Roughness Index, in per mile: Hitch centered, trailer level	Hitch 6 ⅛-in high	Hitch 6 ⅛-in low	Bouncing on trailer hitch during test
1	46	43	44	-
2	45	43	44	44

On the basis of the seven years of tests and experimentation with the University of California roughness indicator, it is believed that if the modifications in the BPR roughness indicator proposed in this paper are made and if all the suggested precautions are taken to eliminate erratic results, consistent results and satisfactory operation of the BPR roughness indicator are reasonably certain to follow.

Dynamic Stability of the Roughness Trailer

Some concern has been expressed concerning the possible variable effects in the roughness measurements which might be attributed to the towing vehicle. The Bureau of Public Roads designed the roughness trailer to be dynamically stable so that the towing vehicle would in no way influence the roughness values measured with the trailer. In Table 10 the results of tests are given for four different towing conditions used to obtain a measure of the dynamic stability of the roughness trailer. The effects of three different heights of the trailer hitch were investigated and the effect also of bouncing on the bumper of the towing vehicle to which the trailer hitch was fastened was investigated. Since the test results were very nearly the same for all four test conditions, it may be assumed that the roughness trailer is dynamically stable.

Design and Use of Outrigger-Trailer Carrier

In the BPR Manual of Information Concerning the Operation and Maintenance of the BPR Roughness Indicator, a truck type towing vehicle is recommended for use with the roughness trailer, so that when the test sections are more than 100 miles apart, it is recommended that the trailer be placed within the towing vehicle when traveling from one test site to another. A suitable tow truck was not available at the University of California when the roughness tests were started and instead of the truck a passenger car was used as the towing vehicle. For the first four years the roughness trailer was towed at all times. It was found that towing the trailer caused a lot of wear on the various bearings and similar parts of the trailer. It also caused uneven tire wear. While the BPR design of the roughness trailer hitch provided excellent universal joint action when towing the trailer, it was not a good design for making a quick change in attaching or detaching the trailer to or from the towing vehicle. To correct all of the above difficulties in running tests with the University of California test trailer, an outrigger-trailer carrier was designed and built in 1953. The design of the outrigger-trailer carrier was patterned after the design of a similar unit built by the Oregon State Highway Department.

A general view of the outrigger-trailer carrier is shown in Figure 13. The trailer carrier is designed with a special spring suspension, electric trailer brakes, a cable hoist and frame to raise and lower the roughness trailer as needed, and with such other features as were needed to facilitate the running of tests with the roughness trailer and for transporting it from one test site to another.

In Figure 14 the roughness trailer is shown clamped in the raised position on the outrigger-trailer carrier. This is the position of the test trailer now used to move it from one test section to another. For running tests the clamps are removed and the test trailer is lowered with the cable hoist to the normal pavement position. The test trailer is attached to the outrigger-trailer carrier with the universal joint hitch shown on the BPR plans. There is no other connection between the test trailer and the outrigger-trailer carrier when it is lowered into position for running the roughness tests. By using a standard heavy duty ball and socket trailer hitch to attach the outrigger-trailer carrier to the towing vehicle, the outrigger-trailer carrier and the test trailer can be quickly and easily detached from the towing vehicle. A special wheel attached to the outrigger-trailer carrier near the hitch can be lowered in position to support the trailer carrier entirely on wheels and to facilitate moving the trailers by hand when they are detached from the towing vehicle.

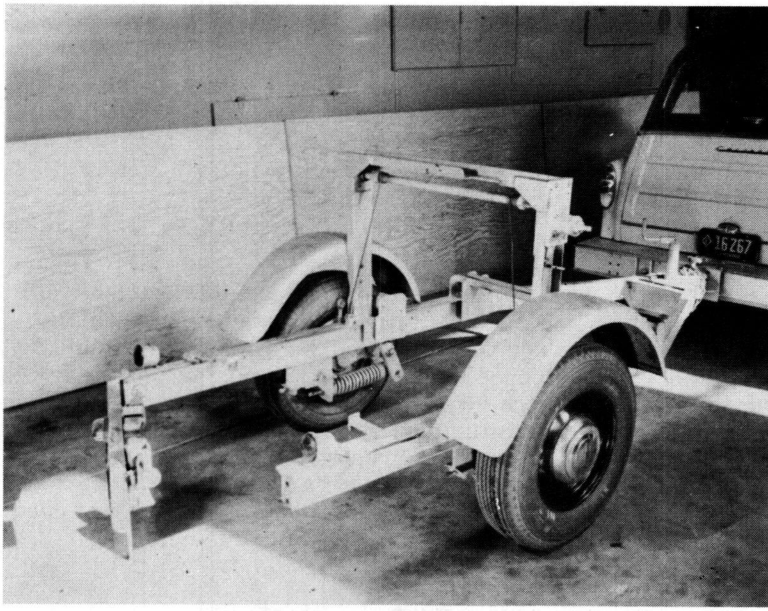

Figure 13. Showing open end of outrigger trailer carrier, trailer spring design, and cable hoist and frame, etc, used for BPR roughness indicator.

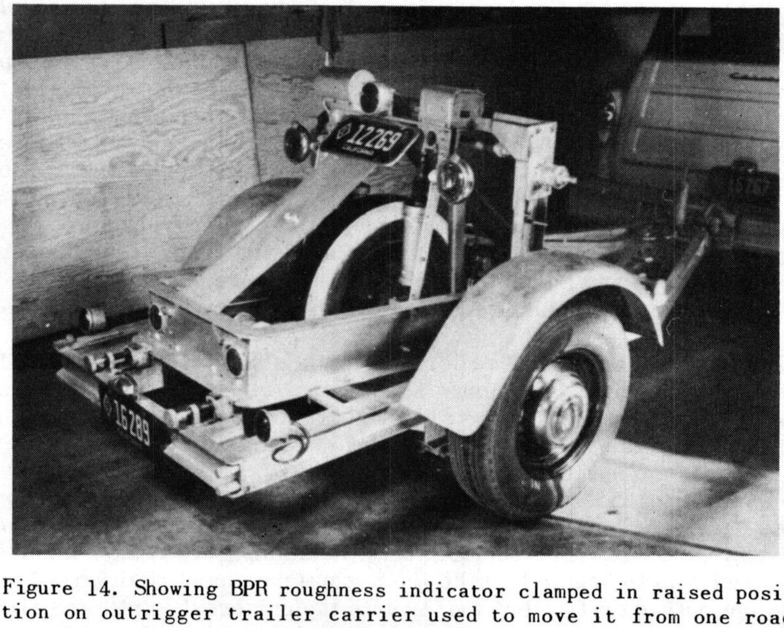

Figure 14. Showing BPR roughness indicator clamped in raised position on outrigger trailer carrier used to move it from one road test section to another.

The use of the outrigger-trailer carrier has eliminated much of the wear on the important working parts of the test trailer and has resulted in the elimination of many of the errors formerly encountered when the test trailer was towed from one test site to another. Detailed working drawings have been made for use in the construction of the outrigger-trailer carrier. A set of the working drawings may be purchased from the Highway Research Board.

RESULTS OF ROAD ROUGHNESS TESTS ON CALIFORNIA PAVEMENTS AND BRIDGES AND ON THE WASHO TEST ROAD

Road roughness tests have been run during the past seven years on many different types of pavements on state highways in all of the eleven state highway districts in California. A special study was made during 1955 of the roughness characteristics of pavements on various bridges and overpass structures in the San Francisco Bay Area. Repeated tests have been run on five sections of pavement selected partly for calibration test purposes and also as the basis of observing seasonal and annual changes in road roughness which might develop on these pavements. Oscillograph records were taken on many sections of pavements to provide a graphical record of road roughness under many different pavement conditions. Oscillograph records of selected sections of pavements will be presented and discussed in this report. Measurements of road roughness were made on all the pavement sections investigated in the WASHO Road Test. The complete report of these measurements is given in "The WASHO Road Test, Part 2: Test Data, Analyses and Findings" Special Report 22 of the Highway Research Board, 1955. Certain typical results of the road roughness measurements obtained on the WASHO Road Test pavement sections will be presented and discussed in this report.

Seasonal and Annual Variations in Road Roughness

The results of the repeated tests on selected pavements which were made to obtain a record of seasonal and annual changes of road roughness, are given in Table 7 and were discussed briefly in the section of this paper dealing with calibration tests. While small variations in the roughness values for all of the pavements are evident in the data given in Table 7, the greatest change in roughness was that obtained for the P. C. concrete pavement on the Eastshore Freeway. The increase in roughness of from 50 to 82 in. per mile for this pavement was due largely to settlement and structural failure of a section of the pavement about 200 ft in length. After resurfacing this section of pavement, the roughness was reduced to 68 in. per mile. On the basis of the roughness standards proposed in the 1951 report, all of the pavements for which data are given in Table 7, except the pavement on Oxford Street in the City of Berkeley, would be rated as excellent. Although small variations in roughness were observed which appeared to be related to temperature and pavement moisture effects on both the concrete and asphalt pavements, the long range effect of these factors could not be clearly established primarily because the BPR roughness indicator during the development stages of the first four years of this study lacked the accuracy and dependability required to determine these effects. Even with the improvements made on the roughness equipment at various times during the past seven years, the data in Figure 7 indicate that the variations of roughness are largely within the experimental error except for the major change referred to above for the roughness of the concrete pavement on the Eastshore Freeway.

The average and maximum roughness values for different types of pavement surfaces on rural state highways in California tested in 1954 and 1955 are shown in Figure 15. In general the lowest roughness values were measured on P. C. concrete pavements with a low value of 40 in. per mile, an average value on new concrete pavements of 66 in. per mile and on old concrete pavements of 88 in. per mile. On one section of new plant-mix asphalt pavement a low roughness value of 35 in. per mile was measured but the average roughness of the new plant-mix asphalt pavements was found to be 85 in. per mile and for plant mix pavements more than two years old, it was 81 in. per mile. The roughness values shown in Figure 15 were for asphalt pavements with seal coats. The roughness values for seal coats less than two years old averaged 80 in. per mile and for seal coats more than two years old, the average roughness index was 94 in. per mile. Under the proposed standard of roughness given in the 1951 report, all of the pavements would be given a rating of excellent although under the Minnesota standard

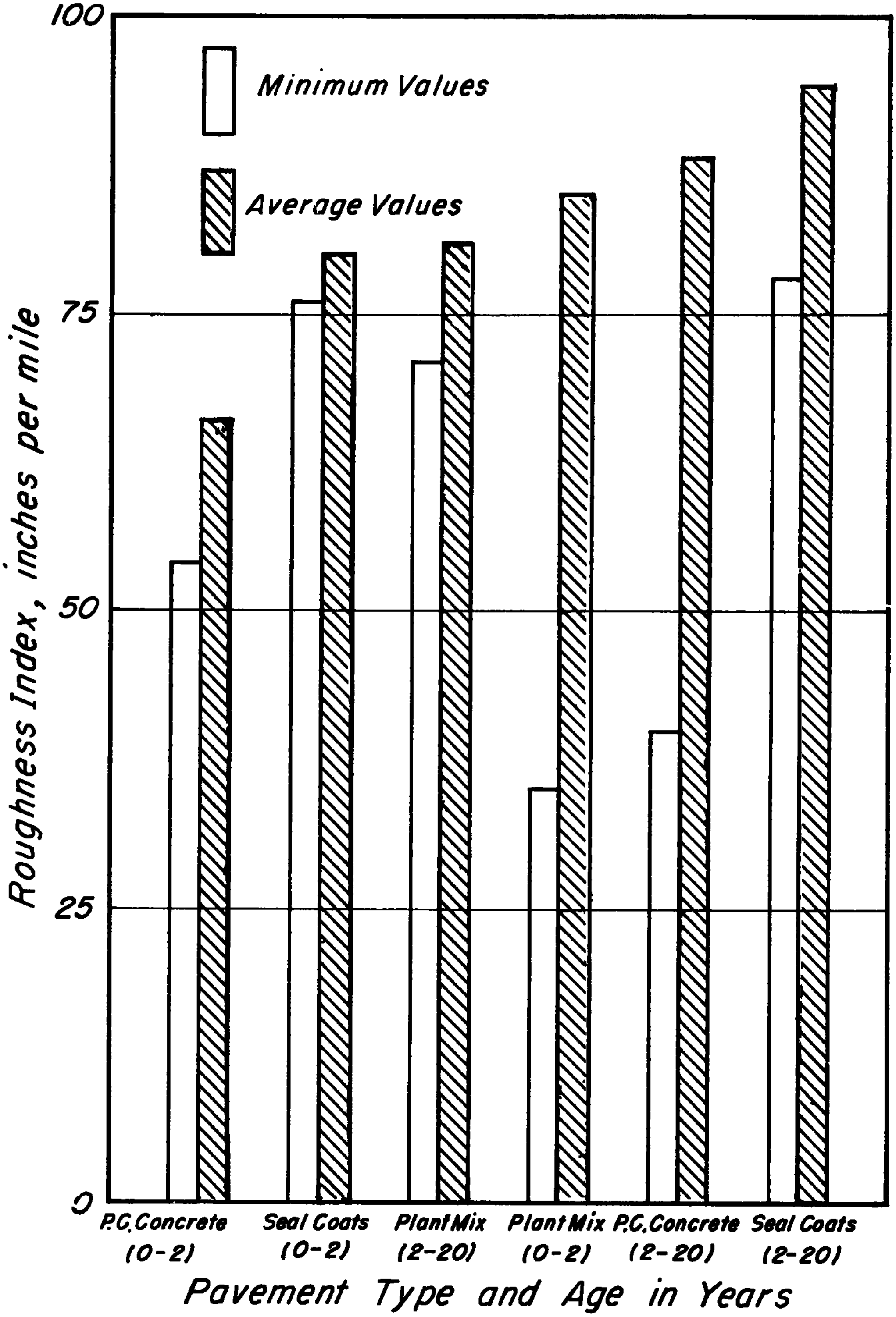

Figure 15. Roughness values for different types of pavement surfaces on rural state highways in California tested during 1954 and 1955.

for new pavements, pavements with a roughness index of 75 to 100 in. per mile are given a rating of fair. On the basis of the Minnesota standard, it may be assumed that certain pavements, notably asphalt pavements with seal coats, for which average roughness values of 80 to 94 in. per mile are given in Figure 15, were not built to an acceptable standard of smoothness.

The roughness of asphalt pavements with seal coats depends primarily on the roughness of the surface on which the seal coat is placed. The high values of roughness measured on seal coats in California is an indication that the seal coats were placed on new surfaces which were not built to an acceptable standard of pavement smoothness. Another consideration is that seal coats are frequently placed on old surfaces which are not patched or repaired to a high standard of pavement smoothness. The use of seal coat construction for filling holes and low spots and for leveling operations is almost certain to result in a rough pavement surface condition and should not be used on pavements where a high standard of surface smoothness or riding quality is desired.

In Figure 16 roughness oscillograph records are shown for certain typical sections

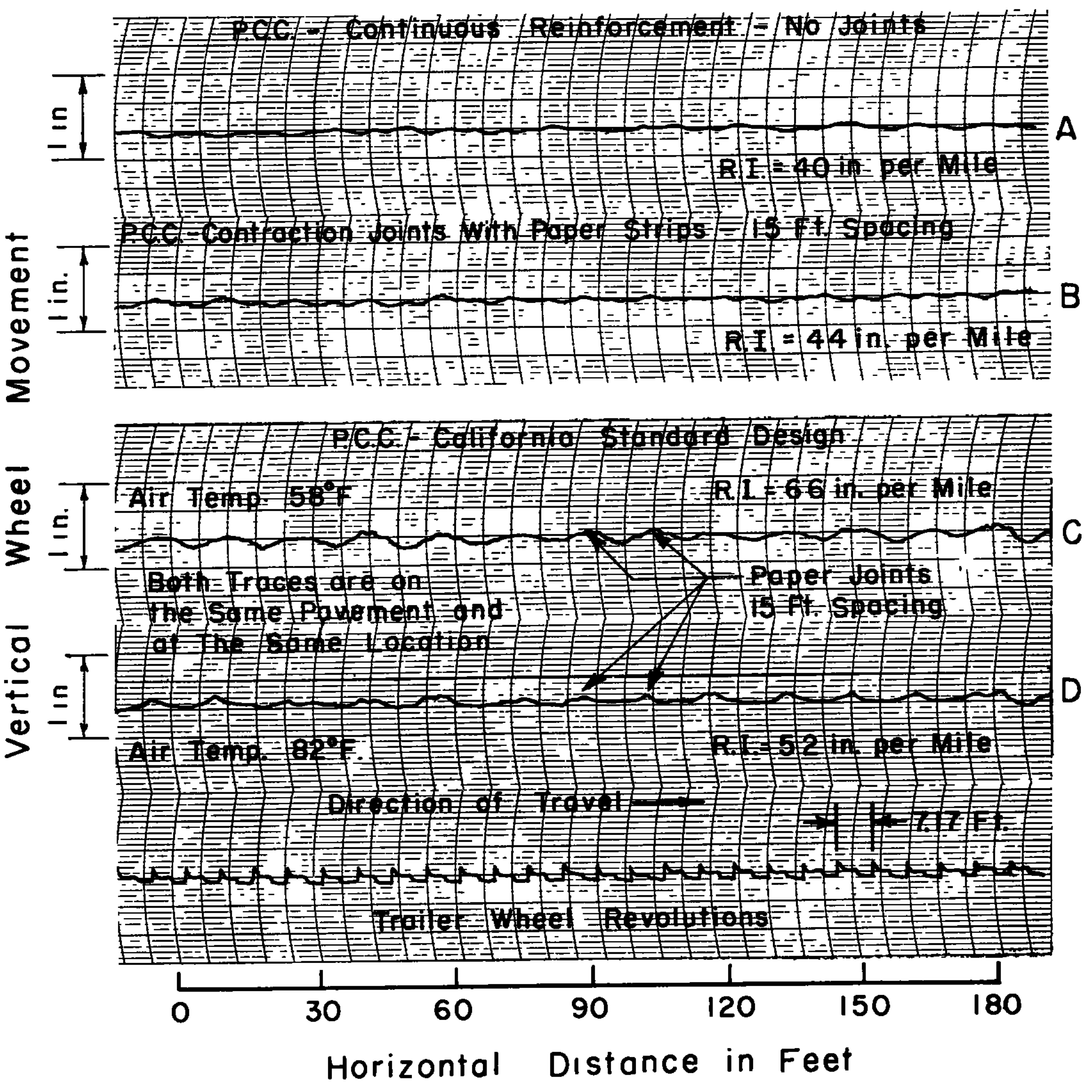

Figure 16. Roughness oscillograph records for P.C. concrete pavement.

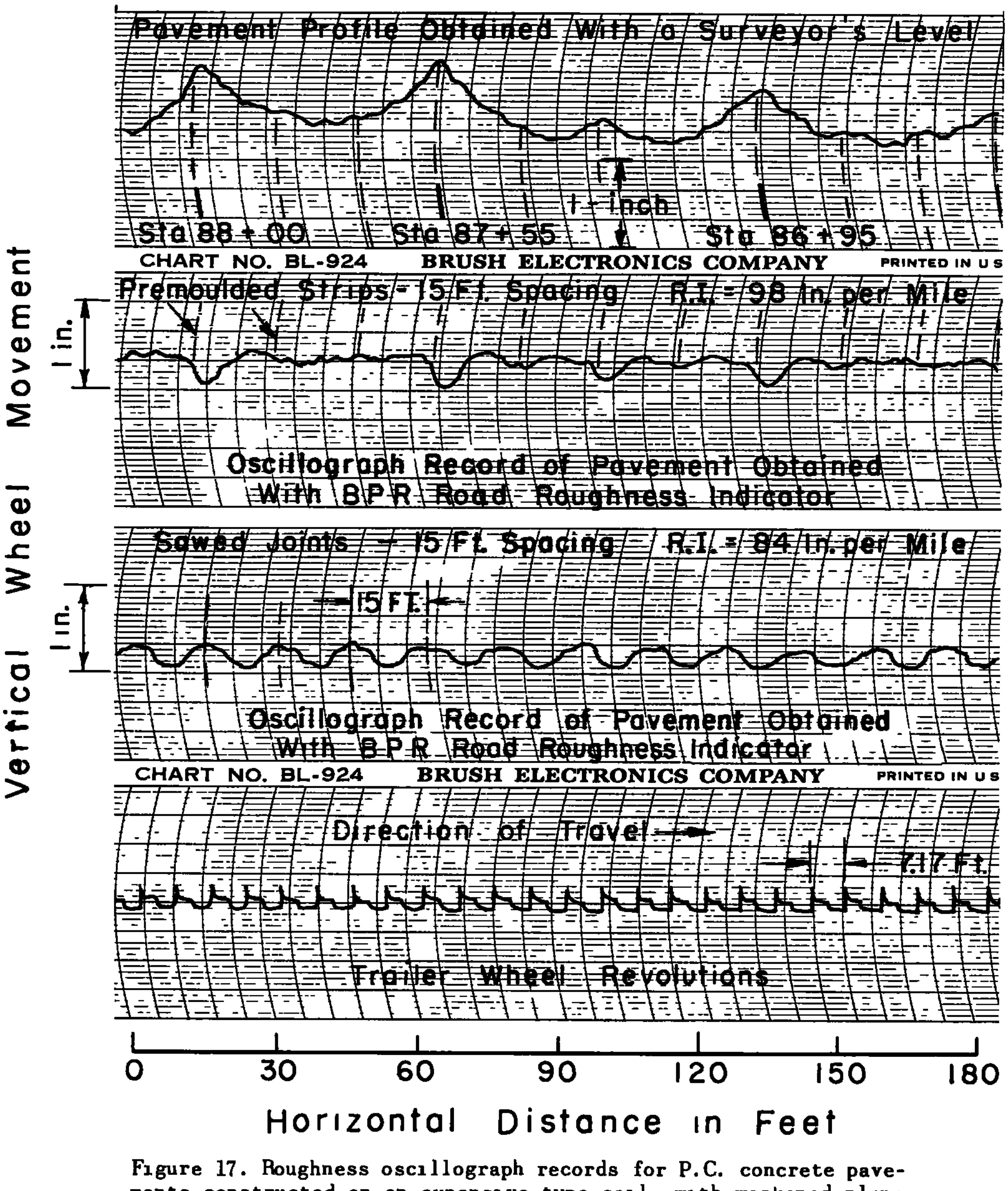

Figure 17. Roughness oscillograph records for P.C. concrete pavements constructed on an expansive type soil, with weakened plane contraction joints of (a) premolded strips and (b) sawed joints. A profile of a section of pavement (a) is also shown.

of P. C. concrete pavements. The records of the sections of pavement with roughness index values of 40 and 44 in. per mile indicate no substantial change in the surface characteristics of smooth concrete pavements construction without joints or with transverse contraction joints spaced 15 ft center to center. The two lower oscillograms in Figure 16 are intended to show the effect of an increase in air temperature on the roughness of concrete pavements. With an air temperature of 58 F, the roughness index was found to be 66 in. per mile whereas by mid-afternoon when the air temperature reached 82 F, the roughness of the same section of pavement was reduced to 52 in. per mile. The oscillograph record for the 82 F air temperature shows improved surface roughness with less warping of the joints than is evident in the oscillograph record for the same pavement with the air temperature at 58 F.

In Figure 17 roughness oscillograph records are shown for a section of P. C. concrete pavement constructed over an expansive type soil using weakened plane contraction joints constructed with thin premolded paper strips for part of the project and with sawed joints for the remaining portion of the project. A profile of a portion of the pavement with the paper joints obtained with a surveyor's level is also shown in Figure 17. Both the oscillograph records and the profile obtained with the surveyor's level indicate the presence of warped joints which contribute to the poor riding quality of these sections of pavement. While the roughness index values of 98 in. per mile and 84 in. per mile are above average for concrete pavements, they are not high enough to give these sections of pavement a low rating. The explanation for the relatively low roughness index values for these two sections of concrete pavement lies in the fact that the roughness was confined to the warping at the contraction joints, while the rest of the pavement was constructed with a smooth finish. Passenger cars with a soft spring suspension system could be operated over the warped joints with fairly good riding qualities but the ride on trucks with stiff springs and a long wheel base was very rough.

Roughness Measurements on Asphalt and Concrete Paved Bridge Floors

The use of finishing machines in the construction of both concrete and asphalt pavements has been an important factor contributing to the low roughness index values measured on concrete and asphalt pavements. In the construction of bridge floors, machine finishing equipment is frequently replaced by hand finishing methods. Also, there are other factors which may effect the smoothness of the paving on bridge floors, especially concrete bridge floors, such as, the method of forming or framing used, the size and shape of the floor panels, the width of the pavement and the provisions made for camber and plastic flow of the concrete.

In Tables 11 and 12 the results of roughness measurements are given for many paved bridge floors on freeway overpass structures and on the major bridges in the San Francisco Bay Area. Oscillograph records of certain typical sections of paving on bridge floors are shown in Figures 18 and 19.

TABLE 11

ROAD ROUGHNESS RESULTS ON P. C. CONCRETE PAVEMENTS ON BAYSHORE AND EASTSHORE FREEWAY OVERPASS STRUCTURES

Eastshore Freeway Overpass Structures	Roughness Index in. per mile
23rd Ave Overpass, northbound (1948)	134
23rd Ave Overpass, southbound (1948)	108
29th Ave Overpass (1948)	106
Fruitvale Ave Overpass	116
Hegenberger Road Overpass	78
98th Ave Overpass	109
Davis St Overpass	80
Albany Overpass (1935)	85
Bayshore Freeway Overpass Structures	**Roughness Index in. per mile**
Bayshore Freeway, elevated structure	
Average Roughness Index - 5th Street to 17th Street	158
Roughness Index - 5th Street to 9th Street	180
Army St Overpass	94
Alemany Blvd Overpass	105
San Bruno Overpass	83
San Francisco Airport Overpass	79
Millbrae Ave Overpass	84
Broadway Overpass	94

TABLE 12

RESULTS OF ROAD ROUGHNESS MEASUREMENTS MADE IN 1955 ON THE ASPHALT AND P.C. CONCRETE PAVED BRIDGE FLOORS OF THE MAJOR BRIDGES AND APPROACH STRUCTURES IN THE SAN FRANCISCO BAY AREA

San Francisco - Oakland Bay Bridge (opened to traffic 1936)	Roughness Index in. per mile
East Bay Crossing, Cantilever Section, P. C. C.	130
Yerba Buena Tunnel and Approaches, P. C. C.	71
West Bay Crossing, Suspension Section, P. C. C.	87
Fifth St Ramp, P. C. C., Average Roughness Index	108
Fifth St Ramp, P. C. C., Average Maximum Roughness Index	130
San Francisco - Oakland Bay Bridge Approaches (opened - 1955)	
Elevated Roadway, West Approach near 5th St, P. C. C.	198
Elevated Roadway, East Approach over Eastshore Highway, P. C. C.	141
Golden Gate Bridge (opened to traffic 1937)	
North Approach Anchor Span, P. C. C.	118
Suspension Section, P. C. C.	110
South Approach Anchor Span, P. C. C.	110
South Elevated Roadway Approach, Near Toll Plaza, Asphalt Paving	97
South Elevated Roadway Approach, P. C. C.	138
South Elevated Roadway Approach, S. E. of Toll Plaza, Asphalt Paving	138
Richmond - San Rafael Bridge - Under Construction (1955)	
West Side Crossing, Girder Spans, P. C. C.	128
West Side Crossing, Truss Spans, P. C. C.	129
East Side Crossing, Girder Spans, P. C. C.	110
East Side Crossing, Truss Spans, P. C. C.	118
Carquinez Bridge (opened to traffic 1927)	
South Approach Viaduct, Asphalt paving over P. C. C.	97
Cantilever Spans, P. C. C.	141
San Mateo - Hayward Bridge (opened to traffic 1929)	
West Side Crossing, Timber Trestle, P. C. C.	145
East Side Crossing, Timber Trestle, Asphalt Paving on P. C. C.	94
Open Grid Steel Bridge Floors	
Park St Estuary Crossing	103
High St Estuary Crossing	99
Mossdale - San Joaquin River Crossing	
Open grid steel bridge decking	127
Concrete in wheel tracks in open grid steel bridge decking	139

The roughness index values of concrete paving on the various bridge structures range from a low value of 78 in. per mile to a high value of 198 in. per mile. The latter value was measured on a new structure opened to traffic in 1955 and has been severely criticized by the riding public as a rough section of pavement. It is interesting to note that the roughness values for paving on structures built 20 years ago such as on portions of the San Francisco-Oakland Bay Bridge, the Carquinez Bridge, and the Albany overpass were less than 100 in. per mile while the roughness values for paving on a large number of recently built structures are considerably in excess of 100 in. per mile.

A variable pattern of roughness for concrete bridge floor paving is indicated in the oscillograph records in Figure 18. The Fifth St ramp on the Bay Bridge has a smooth surface finish but the rythmic corrugations are an indication of the lack of camber in the forming of the floor system during construction and of plastic flow after construction which caused a rough riding surface. There is no clearly defined pattern discernible in the oscillograph record for the concrete paving of the elevated structure with a roughness index of 198 in. per mile. The lack of a pattern in the oscillograph record is an indication of poor workmanship in the form work and in the final finishing operations in the construction of this pavement.

While the number of bridge floors with asphalt paving reported in this study is small, the roughness index values for the asphalt paving are in the range of 94 in. per mile to 138 in. per mile, indicating that the asphalt paving on bridge floors is smoother than the concrete paving. In the usual case the asphalt paving is placed as resurfacing on old concrete paving and, if it is placed with a finishing machine, it should be able to meet a high standard of pavement smoothness or riding quality.

In the construction of the concrete paving on the Richmond-San Rafael Bridge a special finishing machine is being used. The final finish, however, is obtained with a hand-

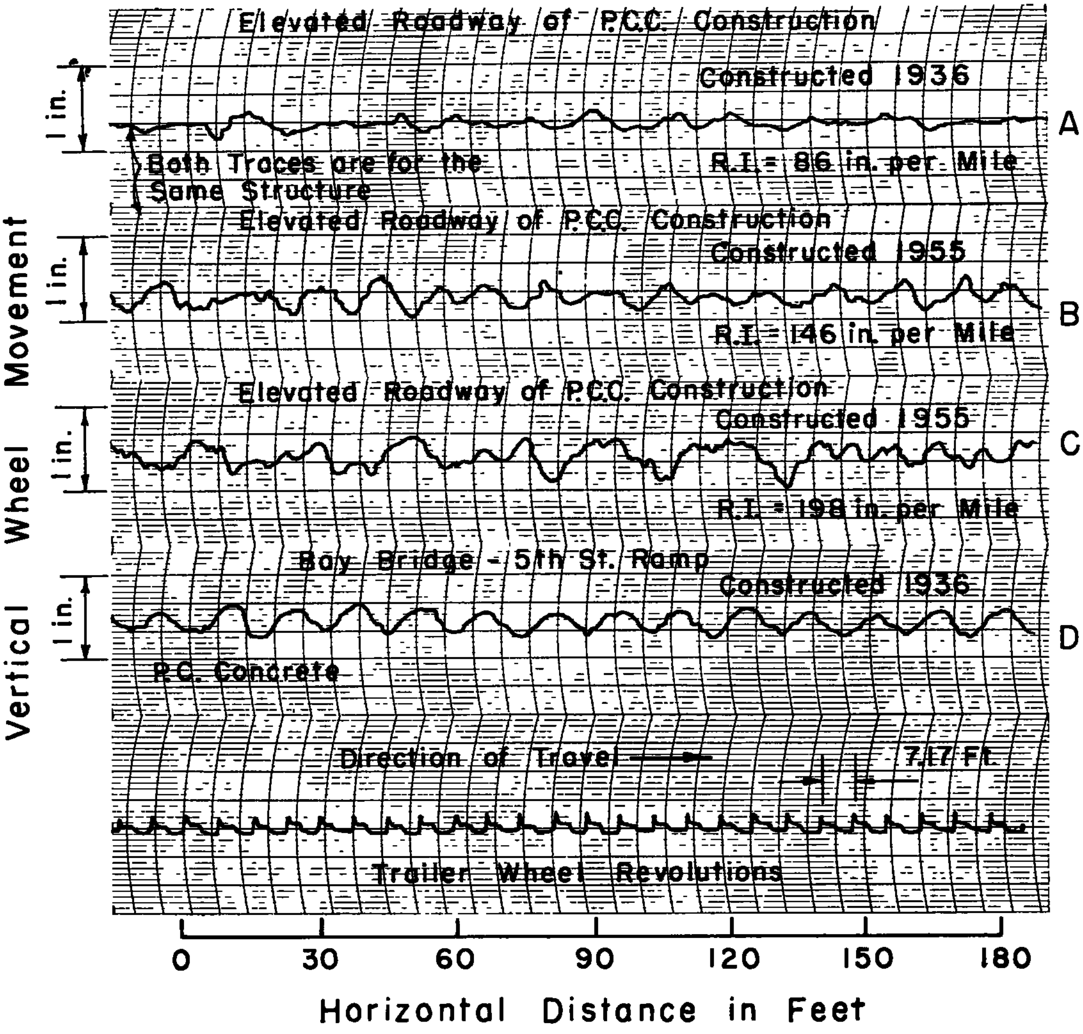

Figure 18. Roughness oscillograph records of P.C. concrete pavements on elevated roadway or bridge type structures.

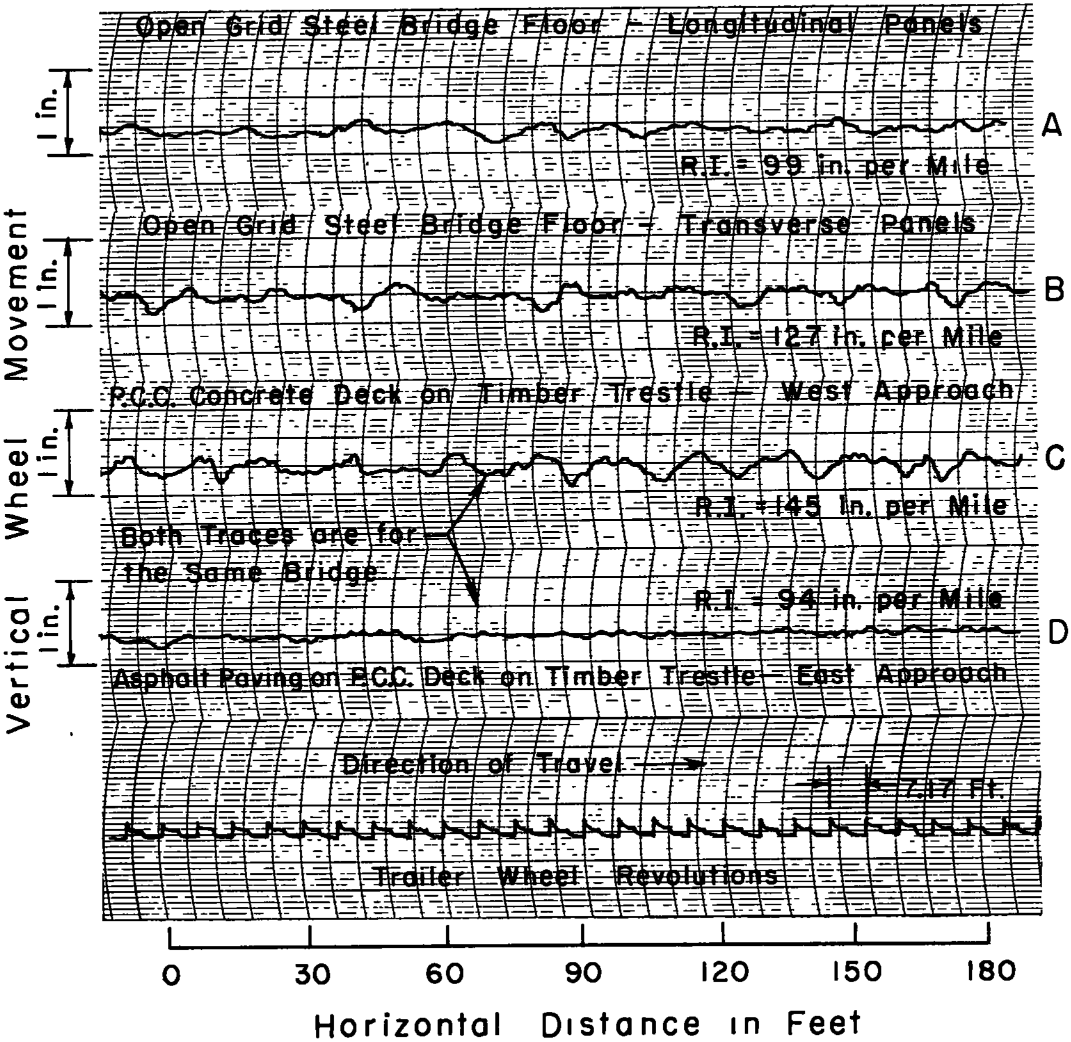

Figure 19. Roughness oscillograph records for four bridge floors.

operated longitudinal float. The roughness index values measured on the completed portions of this bridge vary from 110 in. per mile to 129 in. per mile. It is evident that the concrete paving on this bridge does not measure up to the high standard of smoothness obtained with the Johnson Finisher used on California state highway concrete pavement construction where the roughness index values are usually not greater than 70 in. per mile and they may be as low as 40 in. per mile.

Roughness Measurements on Open-Grid Steel Bridge Floors

The results of roughness measurements on open-grid steel bridge floors are given in Table 12. Typical oscillograph records for two open-grid steel bridge floors are shown in Figure 19. The roughness index values for the open-grid steel decking varied from 99 in. per mile to 139 in. per mile. The oscillograph records indicated that the use of transverse panels resulted in a roughness of 127 in. per mile as compared to 99 in. per mile for a floor system with longitudinal panels. This conformed to the general pattern of smoothness observed on concrete bridge floors in which lower roughness values were measured on floor systems with longitudinal beams and girders than on floor systems with transverse beams and girders or with large open square panels for which proper cambering was generally not provided.

TABLE 13

ROAD ROUGHNESS MEASUREMENTS ON SELECTED SECTIONS OF PAVEMENT OF THE WASHO ROAD TEST. A PARTIAL SUMMARY OF ROUGHNESS MEASUREMENTS MADE WITH THE UNIVERSITY OF CALIFORNIA BPR ROUGHNESS INDICATOR AS GIVEN IN THE WASHO REPORT, PART 2

Section	Test Date	Roughness Index, in. per mile: Outside Wheel Path, 18,000 lb Single Axle	Outside Wheel Path, 32,000 lb Tandem Axle	Inside Wheel Path, 18,000 lb Single Axle	Inside Wheel Path, 32,000 lb Tandem Axle
22-4	6-10-53	92	79	88	84
	11-11-53	82	79	65	70
	6- 3-54	104	88	83	85
14-4	6-10-53	79	75	88	75
	11-11-53	70	66	63	66
	6- 3-54	82	77	71	71
6-4	6-10-53	75	97	75	88
	11-11-53	92	193 (163)	79	114
	6- 3-54	201 (43)	304 (290)	97 (15)	220 (45)
Tangent Average	10-16-52	-	-	85	80
	6-10-53	82	81	82	81
	11-11-53	76	88 (178)	68	81
	6- 3-54	104 (43)	121 (305)	79 (15)	100 (45)

Parentheses indicate the linear feet of patching in wheel path.
Tangent averages include transitions
All roughness values are corrected for trenches and test holes but not for patching.
All tests taken in the direction of traffic.

Road Roughness Measurements on Selected Sections of Pavement of the WASHO Road Test

A partial summary of roughness measurements made with the University of California roughness indicator on selected sections of pavement of the WASHO Road Test is given in Table 13. The complete report of these measurements is given in the Highway Research Board Special Report 22. The measurements indicated that the various sections of pavement on the WASHO Road Test were built to an acceptable standard of smoothness and retained this smoothness until patching was required. The roughness measurements gave no clearly defined indication of progressive structural failure under the repeated heavy axle loads. It is interesting to note that on the 22- and 14-in. sections the effect of traffic was to improve the smoothness of the surface. Thus, the roughness index in one portion of the 22-in. pavement was reduced from 88 in. per mile to 65 in. per mile after being subjected to five months of intensive truck traffic. It is reasonable to expect that the heavy truck traffic tended to smooth out minor irregularities in the surface which were present when the pavement was opened to traffic and that therefore lower roughness index values were measured on these sections of pavement after they were subjected to the heavy truck traffic than prior to the opening of these sections to traffic.

The roughness data in the 6-in. sections show a progressive increase in road roughness although the increase reached significant amounts only after structural failure developed and patching was necessary. Thus in one portion of the 6-in. pavement the roughness index increased from 75 to 92 in. per mile after five months of intensive truck traffic and then seven months later it increased to 201 in. per mile after structural failure developed requiring 43 lineal feet of patching.

CONCLUSIONS

Seven years of tests, experimentation and development work with the BPR roughness indicator have demonstrated that the basic design of this equipment is sound and that it provides the simplest and most accurate method for measuring road roughness which has been developed to date. It is, however, a sensitive instrument which measures displacements as small as five-thousandths of an inch and it is important that all moving parts be built to the same uniform design standards such that the effects of wear, dust and excessive friction or damping effects will be minimized if this equipment is to be used as a standardizable unit for measuring road roughness. Also, special precautions must be taken in the operation and in the maintenance of the equipment to preserve the high degree of sensitivity in road roughness measurements of which it is capable. The use of an outrigger-trailer carrier to transport the roughness trailer from one test site to another is an important factor to protect the equipment from damage and excessive wear.

Roughness measurements on asphalt and concrete pavements in California built with modern pavement finishers have indicated that high standards of pavement smoothness and riding quality can be provided and in general have been provided. The roughness measurements of paving on bridge floors of the major bridges and freeway overpass structures in the San Francisco Bay Area indicate a general lack of acceptable smoothness or riding quality of this type of paving. Improvements in bridge-floor construction methods and in the final finishing operations can and should be developed to raise the riding quality of the paving on bridge floors to the same high standard which is now obtained on highway pavements.

Minnesota Modifications to BPR Roughness Indicator

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During extensive use of the road roughness indicator, built in 1941 from Bureau of Public Roads plans, some modifications of the equipment have proven desirable for the obtaining of consistent and dependable roughness readings. Integrator drive changes and a profile and cumulative tape recorder system have been incorporated. An entirely new integrator built around a commercially available electronic digitizer system is described.

● MINNESOTA'S road roughness indicator was completed and placed in operation in October 1941. It was designed and built on the basis of Bureau of Public Roads plans and specifications revised April, May and June 1941. Operation to date has accumulated over 180,000 miles of travel on two trucks and an estimated 34,000 miles of roughometer recordings. More than 16,000 roughometer recorder miles have been accumulated since 1948. Rivalry among contractors and engineers resulting from its use has been reflected in generally decreasing roughness of new concrete and bituminous paving construction. A low road-roughness index has become so widely sought after since 1946 that it has resulted in increasingly critical interest in operation of the roughometer as well as in construction practices conducive to good riding qualities of road surfaces.

Our use of the roughometer has consisted of the yearly evaluation of construction both as it progresses and following its completion. If the indices reported are to be of value, confidence in them must be maintained. Therefore the use of the road-roughness index as a criterion of roughness requires careful operation of the equipment to assure accuracy and consistency. Furthermore, if comparisons are to be made of the roughness results over a period of years it is necessary to recognize and make proper corrections for the slowly varying responsiveness and freedom of action of the roughometer components.

In view of the wide interest in riding qualities as expressed by the roughness index, modifications of the roughometer which can provide more reliable, positive and generally useful information are desirable. Other additions, which may reduce the necessity for repeat runs due to operator omissions or later need for supplementary information, are likely to reduce the cost of each mile evaluated.

Difficulties Experienced

Almost all operational procedures were standardized in the Bureau of Public Roads Operations and Maintenance Manual of May 1941. However, some inadequacies became evident soon after routine recording started.

During the final stages of construction of our equipment, wartime priorities made standard size drum-drive cable unavailable. It thus became necessary to substitute an equal length of ⅛-in. drill-rod for most of the required length of drum-drive cable. Tests had indicated that stretch in the available drum-drive cable was appreciably impairing the integrator efficiency within the range of the shorter strokes. Integrator calibration had indicated a fairly constant stroke-length loss but to a lesser extent when drill-rod was used in place of part of the cable. Overcoming the static ball-clutch friction at each stroke causes about 0.003 to 0.005 in. stretch of the cable with resultant loss of motion between the axle and integrator drum. Setting the static position cable anchor out an inch is necessary to avoid bending the cable rod and eliminate the attendant efficiency loss.

Extraneous cumulative roughness counts were observed in October of 1941 while determining roughness index on a smooth-riding road. The occasional registering of two to three inches of roughness in abnormally rapid succession followed by the normal time interval to the next count suggested some cause such as arcing at the cumulative count commutator. Laboratory calibration tests at short stroke confirmed this observation and a simple over-center snap-action switch was substituted for the commutator in late 1941 to completely eliminate this difficulty.

The construction division in 1942 felt that the roughness index data as manually recorded was deficient in usable detail. Short-interval recording which avoided operator fatigue and error was originally obtained in 1942 by means of an 8 mm camera. This camera automatically recorded simutaneously the two counter readings showing cumulative roughness and wheel revolutions at intervals of 251 feet. This method did isolate roughness variations down to 251-ft sections but still left much to be desired.

A thorough overhaul of the roughness indicator in 1944 brought to light small indentations which had developed in the spring shackle ball-bearing races. Except during World War II when bronze bushings were used, these bearings have been replaced each spring. The effect of these indentations can be felt when the bearing alone is carefully rotated in the hand, and the restraint, though seemingly slight to the touch, very significantly reduces the road roughness index. Double-sealed and shielded New Departure 99504 bearings have been substituted and are now used without the canvas shackle pants, in order to avoid restraint.

Not noticeable when using the original tire of natural rubber in 1941, but distinctly so in later years when using a synthetic tire, was the change in roughness reading after a "warm-up" run. When transported between distant test locations the roughometer is thoroughly secured within the towing vehicle. Observations have shown that any sustained normal load on a standing synthetic tire will result in a flat spot and extraneous roughness in the roughness index. Securing the road roughness indicator within the transporting vehicle with the wheel clear of the floor eliminates the difficulty and is now standard practice.

Tape Recorder

A continuous recording of profile and cumulative road roughness characteristics had been desirable for some time. Experience of one of the authors in automatic-pilot operation during 1943-5 suggested a logical basis for such a recorder. In 1947 when equipment became available a tape recorder system was assembled. A two-channel magnetic pen-motor oscillograph continuously records a profile and a cumulative graph readable to a fraction of an inch in roughness. This system makes it simple to automatically record pips at established distance intervals and to manually indicate selected locations as desired.

Information made available by such a cumulative tape record permits detailed analysis of any increment of a roughometer run. Such analysis can segregate variations in roughness resulting from changes in construction practices. Exceptions can be made for readings over bridges, railroads or other non-uniformities which occur during a run. A closely tied-in permanent record of roughness for future reference is thus available with pencilled notes placed on the tape during the roughometer run. Avoiding necessity of re-run due to operator omission in manual recording of the location of a detailed and closely tied-in section in itself makes a tape recorder useful.

Profile records also help to indicate whether roughness is general or localized and to pin-point sections which may be especially smooth or rough. High or low joints and other roughness characteristics can be determined from the oscillograph tape. The profile pattern of bump sequence and their magnitude often indicates why some types of roughness are more objectionable than others. Through experience the character of roughness suggests corrective measures in construction and finishing.

Where no detail is called for and a tape is not required, as in some statewide roughness condition surveys, a warning bell indicating mile points has been found to be of value. Its use has eliminated time-consuming reruns necessitated by operator omissions. This bell is actuated by the same circuit previously mentioned which automatically records the one mile pips when the tape recording device is used.

Integrator Calibration

Obvious descrepancies between a high degree of integrator calibration efficiencies and inconsistent roughness indices in the field, led to some changes in 1949. Indications were that integrator calibration as normally done at a repeated fixed length of stroke is not a true index of integrator efficiency on variable stroke, sequence and magnitude as occurs in the field. We were experiencing considerable spring trouble in the internal drum drive. It was believed that the possibility of internal spring drag at variable length of stroke should be avoided. An external drum-drive spring has been used since that time with a torque increase of about five times.

Integrator calibration should indicate a high efficiency but this in itself is no assurance of a high degree of over-all accuracy. Calibration of individual parts of the entire roughometer has been largely abandoned so long as an over-all check is satisfactory. A selected section of near-by high-type bituminous road having lane markers which can be followed accurately for repeatability has been used for approximately seven years as a standard for calibrating or determining the reliability of the roughness indicator. Another road is occasionally used for double-checking. During the active roughometer season a check on the standard test site is generally made each time the unit returns to the laboratory. The reassurance is simple and rapid and reasonably repeatable, but some type of steel-track check-run would be very desirable.

Frequent interstate comparison of equipment and roughness indices concurrently run on the same roads would be highly desirable. To compare indices between states without such comparison is not realistic. Comparison of spring deflection data, integrator calibration and interchange of integrators, damping cylinders and other parts might do much to standardize equipment, operation and roughness measurements.

There are possibilities of wide variation in roughness results. Such inconsistencies are a function of the operator's alertness to variations in the mechanical condition of the equipment with respect to freedom or restraint of movement during test runs.

Integrator performance over a period of years must certainly vary, but to what extent is debatable and largely unknown. When a simple reassembling of an integrator results in an appreciable calibration change for the better there certainly is reason for concern. Nevertheless there are probably few who can point out where and when repair or replacement are necessary. This was the situation which caused us in 1954 to search for a checking or companion integrator.

Electronic Integrator

Our present electronic integrator is an adaptation of Telecomputing Corporation's Magnetic Shaft Position Digitizer. The use of this equipment fits commercially available components into a use for which they were specifically made.

Essentially this system divides a cable-drum stroke into minute discrete bits such that they can readily be accumulated by an electronic counter. A radio-frequency, alternating current passes continuously through one winding and magnetically induces an opposite current in a matching winding immediately opposite and facing it. One of these windings is mounted on the stator and one on the rotor of the reading head and are wholly independent of each other mechanically except for a concentric shaft.

Relative motion of the two windings modulates the radio-frequency current which, when returned, to the demodulator, is converted into one discrete pulse for each passing of one winding across the other. Mechanically, the rotor is driven by a cable drum and return spring connected to the roughometer shaft in the same manner as is done with the mechanical integrator. Pulses are produced in both directions and are totaled to indicate the sum of all vertical motion so that identical cumulative indication should be produced by both the mechanical and electronic integrators.

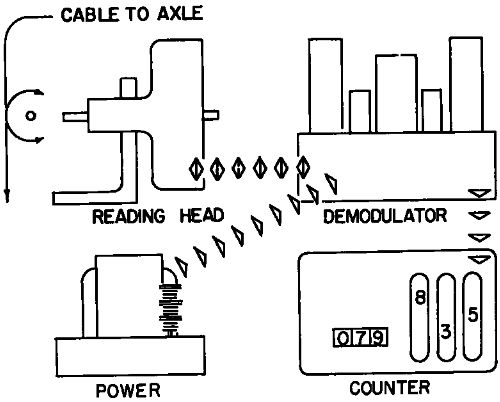

Figure 1. Block diagram of electronic integrator.

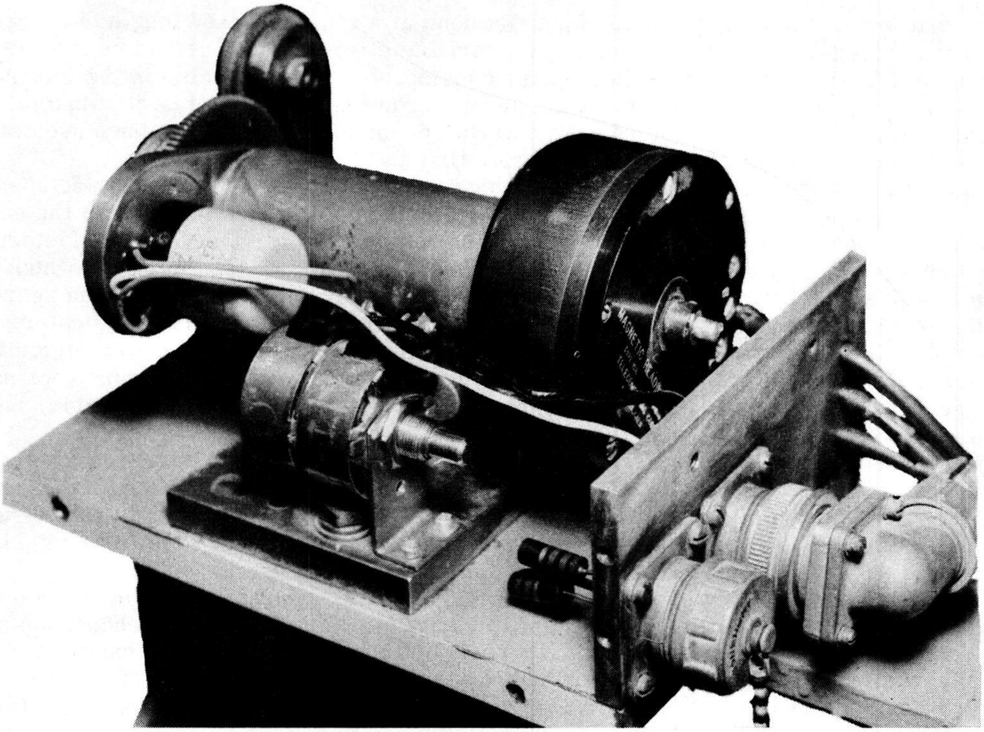

Figure 2. Magnetic reading head.

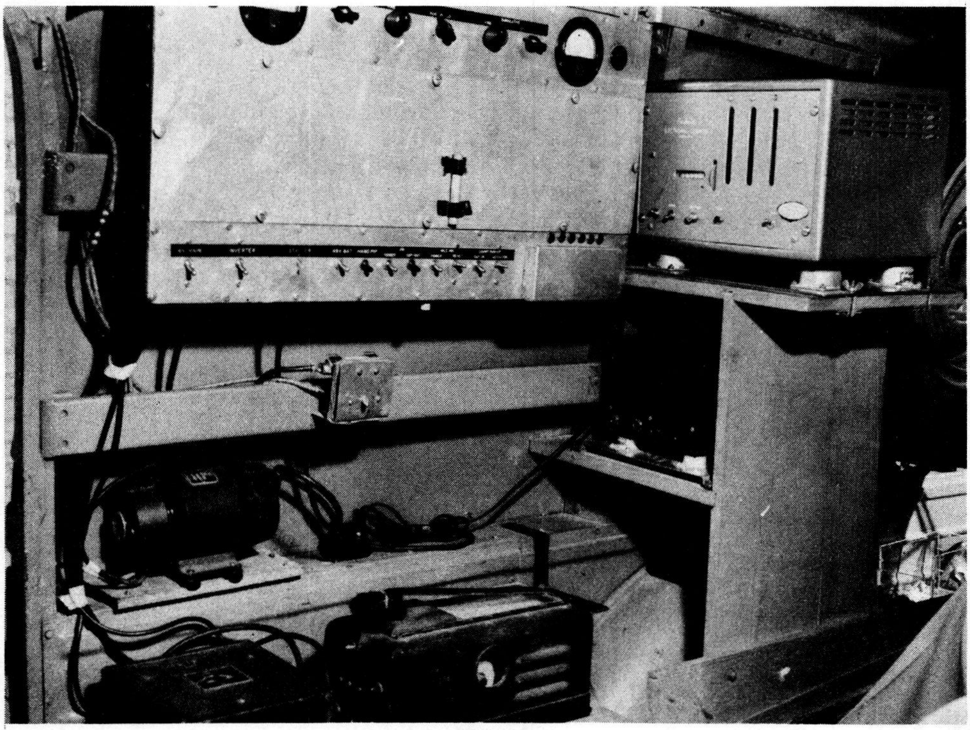

Figure 3. Picture of assembled units as mounted in the body of panel truck.

Four commercially available, electronic units are interconnected to form the integrator and roughness indicating register. These units are shown in the block diagram in Figure 1 and are described as follows:

1. A magnetic, reading head manufactured by Telecomputing Corporation functions to digitize the cable drum rotation. It may be designated as the integrator proper in that it occupies that position on the roughometer. It is mounted in a shop-made cradle which carries the cable drum and profile potentiometer. This is pictured in Figure 2.

2. A small demodulator manufactured by the same company to furnish 1.6 megacycle radio-frequency carrier current to the reading head, demodulate its output and to furnish the resulting pulses, at a rate of up to 60,000 per second, to an electronic counter.

3. A small power supply unit also manufactured by Telecomputing Corporation for the demodulator described above. 110 V AC is required for this unit.

4. An electronic counter manufactured by Berkeley Scientific Corporation accumulates the demodulator output pulses. It has three decimal-counting units and a mechanical register capable of accumulating up to 10,000 pulses per second, random or constant rate. Each thousandth pulse actuates the mechanical register and, a pip on the recording tape previously described, to indicate inches of bump. The assembled units are shown in Figure 3, as mounted within the truck body. The control panel for both the mechanical and electronic integrators is shown in the upper left. The Berkeley counter is pictured in the upper right and the inverter to convert 6V (DC) to 110V (AC) current at the lower left. The power supply and demodulator are mounted on the shelf below the Berkeley counter.

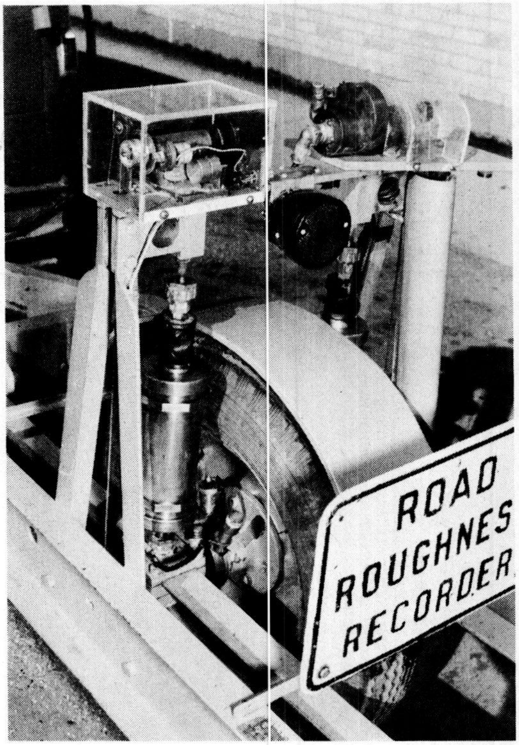

Figure 4. Integrators mounted on roughometer. (Electronic integrator on left, mechanical on right.)

Figure 4 shows both of the integrators mounted on the frame of the roughness indicator.

Performance

No internal friction, other than in the shaft bearings of the reading head, is encountered in driving the electronic integrator. This eliminates from consideration ball-clutch resistance which probably causes the constant cable stretch loss in the mechanical integrator. The inertia of rotating parts is probably less than one fourth of that in the ball-clutch and other rotating parts in the mechanical integrator. There is no such thing as unaccountable ball-clutch slippage in the electronic unit.

As operated during the past season the cable drum and reading head are so connected as to produce 250 pulses per inch of roughness or deviation from a plane. Additional circuitry is available to increase the pulse rate to 500 or 1,000 per inch of roughness. By appropriate Berkeley counter modifications each pulse rate can be made to indicate inches of roughness directly and record on the tape one pip at each inch of roughness.

With both integrators operating simultaneously from opposite ends of the roughometer axle the electronic integrator index has been consistently higher than the mechanical integrator throughout the past summer.

The average of roughness indices in 1955 of 205 miles of concrete pavement shows the electronic integrator to read an average of 2.4 in. per mile greater than the mechanical. The difference varies normally from two to four inches higher with no consistent relationship to the magnitude of the index. Recently when differences ranged up to 21 in. per mile the mechanical integrator was brought back into agreement simply by reassembling.

On the basis of operation to date it is evident that electronic integration can be more consistent and reliable than the mechanical. The equipment is more easily obtained or replaced since commercially available components are used.

Maintenance

Maintenance to date has been nil. When needed it is believed that either replacement or commercial repair services should eliminate the necessity of going along with slowly deteriorating equipment where faulty parts or malfunctioning cannot be pinpointed.

Skid Resistant Pavements in Virginia

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Virginia Council of Highway Investigation and Research

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

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

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

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

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

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

where

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

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

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

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

A Major Cause of Slipperiness

Since the 1947 report, as will be brought out later, it has been indicated that the polishing action of traffic on pavements constructed with limestone aggregates is a major cause of poor skid resistance, even when there is no excess of asphalt and even when the mix has a fairly coarse and open texture. More recent tests have found quite definitely that, except for those surfaces glazed by excess asphalt, the great majority

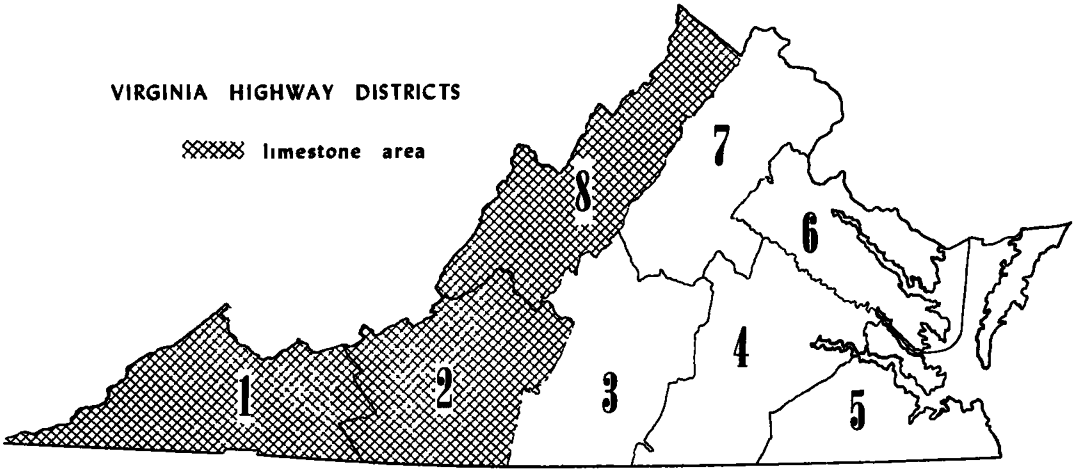

Figure 1.

of slippery pavements in Virginia were constructed of limestone. This situation has become one of great concern to highway engineers in Virginia, in view of the fact that in three of the eight construction districts in the state, practically all of the aggregates quarried are of a limestone or dolomite character.

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

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

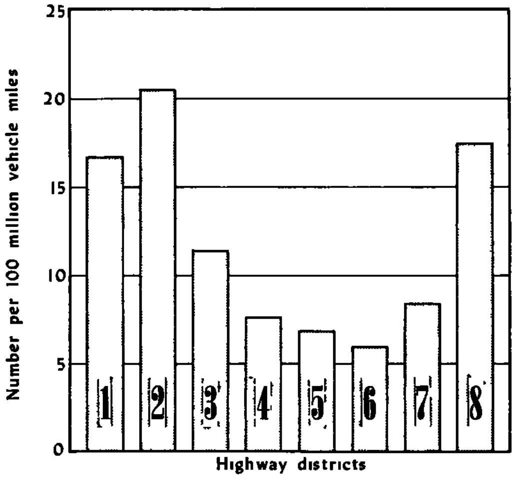

Figure 2. Skidding accidents.

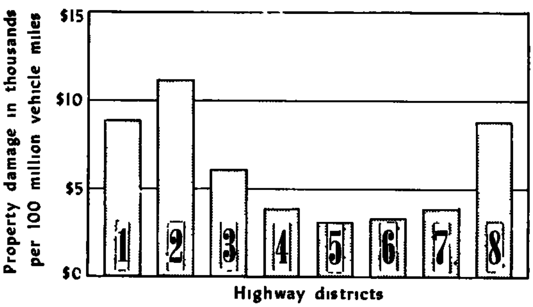

Figure 3. Skidding accidents.

TABLE 1[a]

VIRGINIA ACCIDENT DATA, 1953-1954

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

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

other five, when related to total vehicle miles traveled.

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

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

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

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

Part I
Recent Skid Testing in Virginia

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

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

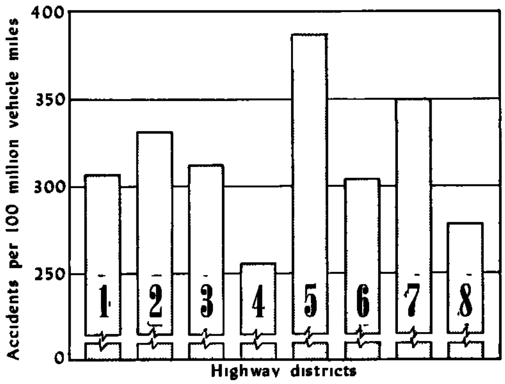

Figure 4. All accidents.

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

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

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

TEST METHODS

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

Stopping Distance Method

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

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

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

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

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

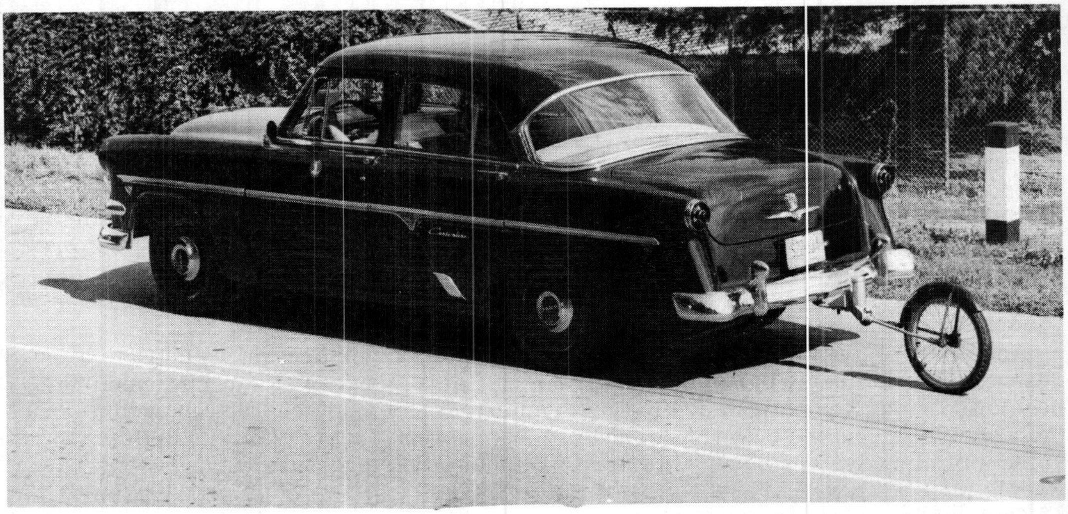

Figure 5.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

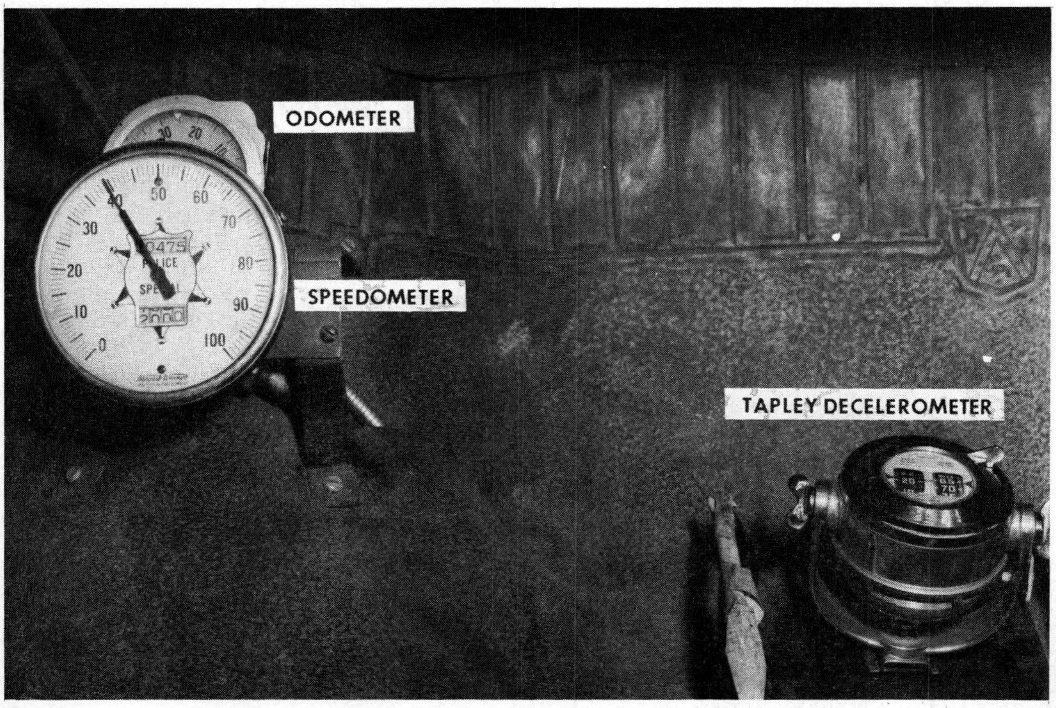

Figure 6.

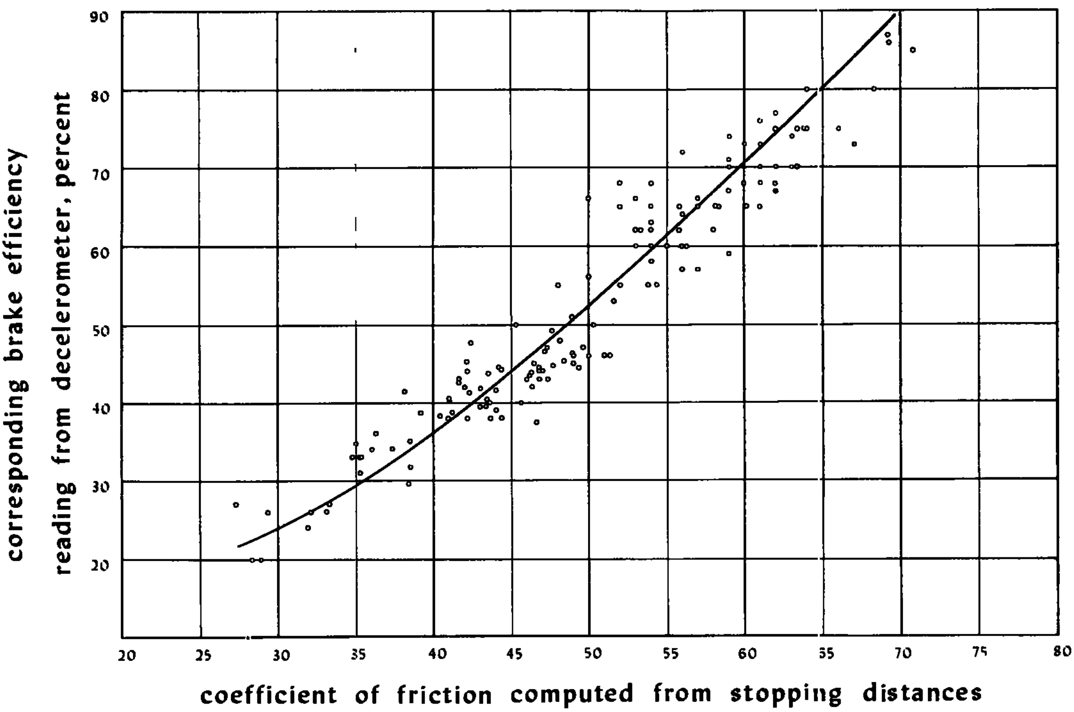

Figure 7. Coefficients of friction versus decelerometer readings on same test run.

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

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

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

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

TABLE 2

SUMMARY OF SKID RESISTANCE DATA BY MAJOR AGGREGATE TYPES

(All Tests from 40 mph on Wet Pavement)

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

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

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

RESULTS OF SKID RESISTANCE TESTS

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

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

TABLE 3

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

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

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

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

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

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

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

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

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

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

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

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

Realizing then that pavements constructed of limestone do result in significantly longer stopping distances than those of any other aggregate common to Virginia, the question naturally comes up: "What should be considered a reasonably safe stopping distance?"

CRITERIA OF ACCEPTABLE MAXIMUM STOPPING DISTANCE

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

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

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

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

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

Part II
Experimental Investigation of Measures for Increasing Skid Resistance

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

PURPOSES OF EXPERIMENTS

The specific objectives of the experimental program were twofold. The first objective was to find the most economical method of building-in a permanent skid resistance at the time of construction. The second was to develop an economical method of deslicking existing pavements whose only deficiency was the lack of adequate skid resistance.

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

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

BUILDING-IN HIGH SKID RESISTANCE

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

Adding Polish-Resistant Fine Aggregate

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

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

TABLE 4

1954-1955 EXPERIMENTAL PROGRAM - PROVIDING SKID RESISTANT ROADS

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

TABLE 5

SKID TEST RESULTS ON MIXES CONTAINING POLISH-RESISTANT FINE AGGREGATE

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

TABLE 6

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

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

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

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

Adding Polish-Resistant Coarse Aggregate

Subsequent to the extensive testing of the limestone pavements containing 20-25 percent silica sand, it was decided that a different approach to building-in skid resistance was necessary. It was assumed that possibly the ineffectiveness was due to the position of the sand within the mix; the tires were riding on the coarse limestone particles rather than on the sand. It was thought, therefore, that the addition of a polish-resistant coarse aggregate[1] might provide a better solution. To determine whether or not this was true, three test sections were placed in 1955.

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

TABLE 7

SKID TEST RESULTS ON MIXES CONTAINING POLISH-RESISTANT COARSE AGGREGATE

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

TABLE 8

SKID TEST RESULTS ON DESLICKING TEST SECTIONS

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

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

Thin Plant Mix Sand Application

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

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

DESLICKING MIXES

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

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

Experimental Mixes of 1955

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

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

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

TABLE 9

GRADINGS OF SANDS USED IN EXPERIMENT

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

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

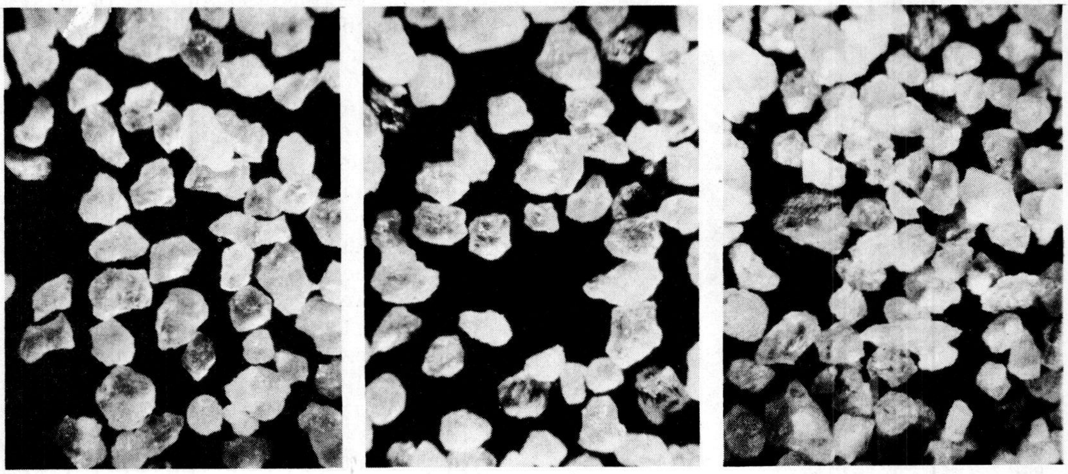

Figure 8. Sand grains in three test sections.

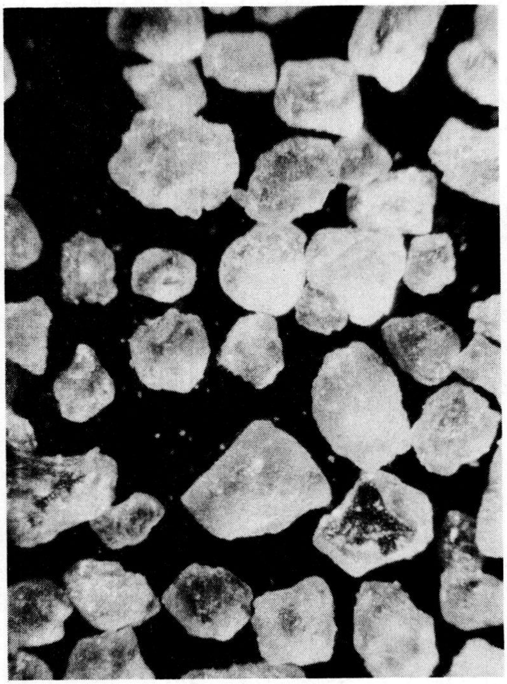

Figure 9. Sand grains in rock asphalt.

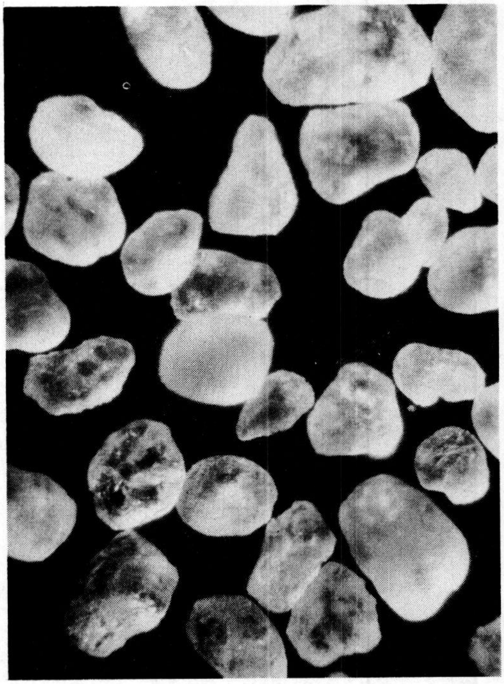

Figure 10. Ottawa sand.

Methods of Application

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

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

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

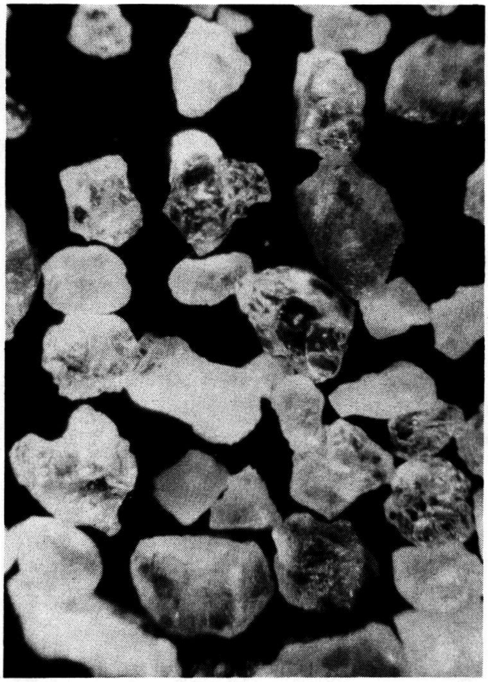

Figure 11. A commercial Coastal Plains sand.

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

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

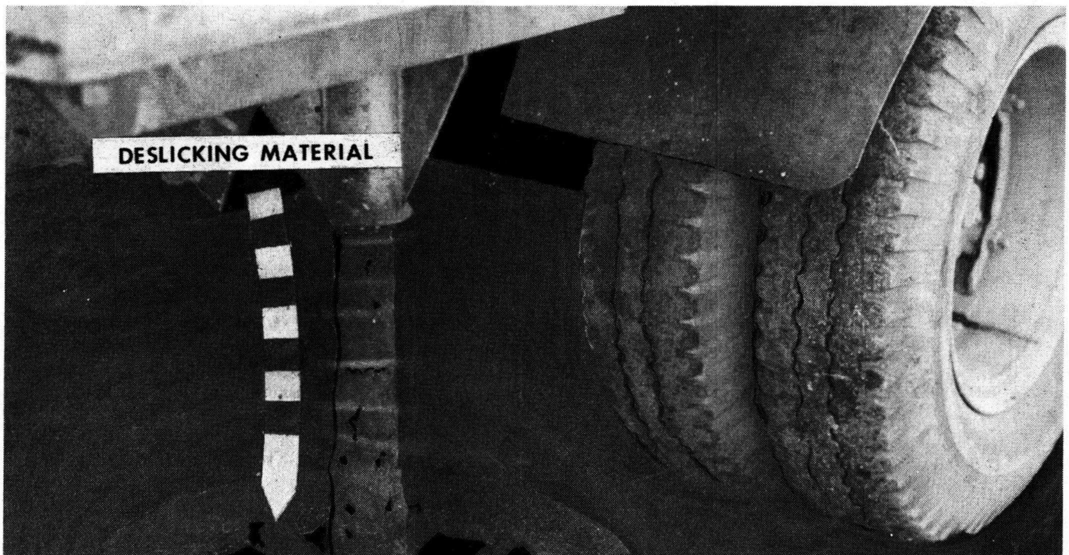

Figure 12.

Figure 13.

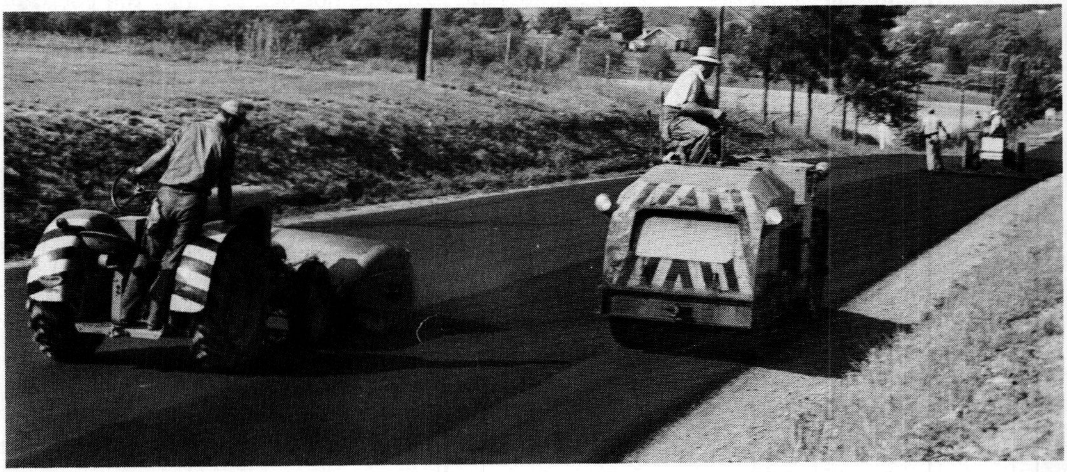

Figure 14.

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

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

DISCUSSION OF EXPERIMENTAL MIXES

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

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

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

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

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

CONCLUSIONS

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

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

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

<u>1950 Experiments</u>

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

4. Rock asphalt (sandstone type) provides excellent deslicking material.

<u>1955 Experiments</u>

5. The "built-in" test sections including coarse and fine polish-resistant aggregate as a part of the asphaltic concrete aggregate blend and the machine-laid thin sand mix cannot be evaluated as yet.

6. Skid test results indicate that the addition of 20-25 percent silica sand to I-3 type mix will not consistently provide adequate skid resistance.

7. Laboratory tests indicate that deslicking mixes made from fine sand utilizing penetration grades of asphalt can be designed with high cohesion and high moisture resistance.

8. Field tests indicate that the plant-made deslicking mixes applied at rates of 10-15 psy can be expected to provide a suitable and economical method of eliminating slipperiness.

FURTHER RESEARCH

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

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

1. The testing of blends of silica sand and limestone that will yield a finer (sand asphalt type) surface texture than the presently used I-3 asphaltic concrete.

2. The testing of sands coarser than those used experimentally for deslicking purposes. In addition, a more detailed investigation of the decelerometer method, with statistical analysis of data, will be undertaken.

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

1. More detailed study of the effects on slipperiness of such factors as grade, tire tread and composition, pavement temperature, and the presence of films of oil, dust, or excess bitumen.

2. Correlation of work done in different states, aiming at the development of a standard method of test so that fairly accurate comparison of results may be made.

ACKNOWLEDGEMENTS

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

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

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

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Appendix

Notes on Virginia Geology and Aggregate Sources

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

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

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

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

TABLE A

SUMMARY OF SKID RESISTANCE DATA BY INDIVIDUAL AGGREGATE SOURCES
All Tests Made from 40 mph on Wet Pavement

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

TABLE A (Continued)

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

6. Arlington Stone Company: and Fair-

TABLE B

TABLE OF AGGREGATE GRADATIONS SPECIFIED FOR VARIOUS TYPES OF BITUMINOUS CONCRETE

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

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

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

Discussion

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

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

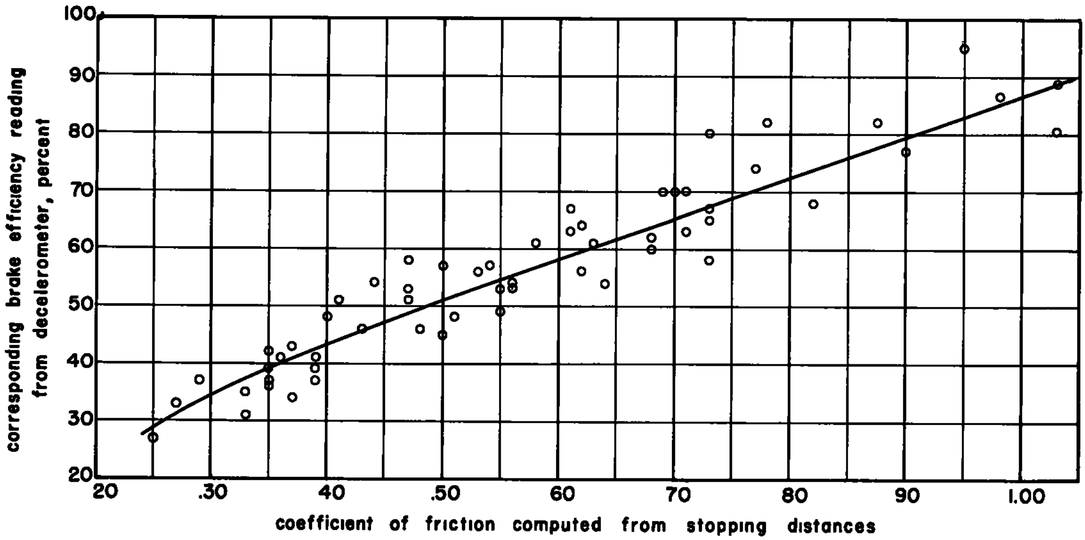

Figure A. Coefficients of friction versus decelerometer readings on same test run.

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

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

Tests involving both the stopping-distance method and the decelerometer method have been performed on 20 different pavement sections. Efforts were made to run the tests from initial speeds of 10, 20, 30 and 40 mph. It soon became apparent, however, that the reproducibility of decelerometer reading for tests conducted from an initial speed of 10 mph was very poor. Such tests have, therefore, been discontinued.

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

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

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Development of Skid Testing in Indiana

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D. L. GRUNAU, Graduate Assistant
Joint Highway Research Project, Purdue University

Many studies have been made in recent years with various types of skid equipment to evaluate skidding characteristics of pavement surfaces. This paper briefly summarizes the equipment used and the results found in these studies and presents a detailed description of a semi-automatic braking device used on a conventional automobile in Indiana.

The device is electrically operated and when activated applies the brakes and initiates measurement of stopping distance simultaneously. The speed at which the brakes were activated is also recorded. The method used eliminates much of the human variable from the measurement of stopping distance and makes it possible for the good reproduction of stopping distance.

The skid testing program in Indiana is also outlined and preliminary results are presented. A number of experimental surfaces were tested along with four major surface types used in Indiana. These four were rock asphalt, portland cement concrete, bituminous concrete, and other bituminous surfaces. A total of 233 different roads were tested; each road being tested at three locations with two skids being performed at each location.

The skidding properties of the various roads were compared in terms of mean skid distances at 30 mph. Variability of the skid distances was determined along with the means.

The tests showed that rock asphalt had the best skidding properties of all the surfaces tested with respect to both average distance and variability. Its mean skid distance changed little between the wet and dry condition. Portland cement concrete surfaces provided relatively good skid characteristics but were subject to some polishing by traffic during the first few years of their life. The bituminous concrete surfaces tested had poorer skid characteristics than any other major type considered. The bituminous surfaces tested, other than rock asphalt and bituminous concrete, had a relatively low mean but were extremely variable. This variability was almost invariably associated with bleeding. Those roads with no bleeding yielded a mean 18 feet less than those that displayed some bleeding. The bituminous roads constructed with limestone aggregate had a lower mean than those containing gravel, although the limestone in some cases polished extensively under prolonged heavy traffic.

● IN 1954, drivers in the United States became involved in 30,800 fatal and 1,276,700 non-fatal accidents. Many of these accidents were primarily the fault of the drivers involved, but a great number might have been minimized or prevented by safer highways. Although many factors are involved in the building of safety into highways, one of the more important items is the resistance of pavement surfaces to skidding, especially when these surfaces are wet. Seventeen percent of the fatal and nineteen percent of the non-fatal accidents in 1954 occurred on wet pavements. When consideration is given to the fact that pavements are wet for less than nineteen percent of the time, and travel is usually reduced during wet periods, it is evident that a disproportionate number of accidents occur under these conditions. How many accidents could be avoided by better skidding characteristics is unknown; but as many accidents involve some type of skidding, the number is undoubtedly of considerable magnitude.

In Indiana, it is generally recognized that certain types of pavements have better skidding characteristics than others, but few measurements have ever been taken on a comparative basis. Experimental sections have been constructed in the past few years, and several new surface types are being used. Some of these surfaces are quite economical and durable, but little is known of their skidding properties. If the new surfaces are dangerously slippery the reduced construction cost is of extremely dubious value, unless they can be redesigned so as to make them satisfactory.

Only a few organizations or individuals have undertaken extensive programs of research to determine the skidding characteristics of pavements. Possibly the most complete data have been obtained by Professor R. A. Moyer, formerly of Iowa State College and currently at the University of California in Berkeley. In his earliest tests, as reported in 1933, a two-wheel trailer towed by a water truck was used. The trailer was constructed so it could be used to measure impending skid, straight locked-wheel skid, and side skid. The skidding force was measured by integrating a dynamometer linkage to the towing truck. The wheels were locked or braked with Bendix self-energizing mechanical brakes which were manually operated. Two to four runs were made in each direction, wet and dry, at 3, 5, 10, 20, 30, and 40 mph and the dynamometer force was averaged over a distance of from 50 to 150 feet.

One of the first detailed reports on another method of test, the automobile stopping-distance method, resulted from work conducted in Virginia by T. E. Shelburne and R. L. Sheppe, reported in 1948. Here a standard, light-weight automobile was used with manually operated brakes. The skid distance was measured by taping the distance from a chalk mark fired from a device mounted on the running board. The roads were compared by computing the coefficients of friction at speeds of 10, 20, 30, and 40 mph. The coefficient of friction was computed by the standard formula $F = \frac{V^2}{30S}$, where F equals the average coefficient of friction, V equals the initial speed in mph at the time of applying brakes, and S equals the average stopping distance in feet. This formula has been used for almost all subsequent tests that have employed the stopping distance method.

Other methods of test, such as a motorcycle sidecar used in England and a trailer with pivoted wheels used in France, have been reported and numerous conclusions from these skid studies have presented valuable information on skid characteristics. Much remains to be done in this area, however, and in order to obtain estimates of the skid characteristics of Indiana road surfaces and to investigate some of the factors that affect skid resistance a long-range study was undertaken.

Research on skid-resistance was initiated by the Joint Highway Research Project in November, 1950. At that time John F. McLaughlin presented a "Report and Annotated Bibliography on Skid Resistance." In 1951 field observations of driver reaction and reliability of test equipment were made. From June 1952 to the summer of 1953, further tests, using the automobile stopping-distance method, were conducted by John Baerwald. The primary purpose of these tests was to develop testing procedures and to provide data for comparison of the skid resistance qualities of pavement surfaces. From these preliminary tests a formal field study was determined to be advisable.

DEVELOPMENT OF A PROGRAM

Testing Device

A preliminary study was conducted in order to determine the testing method that would be the most satisfactory and that would be safe and economical. The vehicle stopping-distance method was determined to be the best for this study even though the towed-trailer method was found to have some advantages. The latter method is certain to give very accurate results as it uses sensitive instruments and results are averaged over a considerable length of pavement. Tests can be run fast and safely as the unit is self contained and does not require that traffic be stopped. The towed-trailer method, however, has definite disadvantages. It is expensive to build; there is a possibility of differences in skidding characteristics between a towed object and a freely skidding unit; and there is some doubt that the "hot spot" developed underneath one wheel of a towed trailer as it is dragged over a long distance will give performance characteristics similar to those obtained from four locked wheels skidding over a shorter distance. Finally, several investigators have indicated that there is a difference between the skidding resistance of a pavement in the initially wet stage and of one in a flushed condition. This presents a possibility that wetting a surface at high speed immediately before a skidding wheel might give unrealistic results.

Because of these considerations the Joint Highway Research Project adopted the basic concept of using the stopping distance of a freely skidding vehicle to compare skidding properties of pavements. In its original form this method of test consisted of a standard 1951 Ford equipped with a chalk-marking device mounted on the rear bumper, the wiring being attached to the brake pedal by a large clamp connected in series with a simple pull-apart connector. To run a test the driver would connect the proper wiring and load the chalk marker with a chalk cartridge. He would then bring the vehicle to a speed slightly in excess of the test speed, disengage the clutch, let the vehicle coast to the proper speed and slam on the brake, bringing the car to a skidding stop. The movement of the brake pedal fired the chalk marker, and the distance from the mark to the final position of the car was taped.

It was felt at the outset of this survey that this method in its initial form was too crude and presented several disadvantages. Some of these were:

1. The driver may differ in his reaction time from skid to skid and day to day.
2. The driver may differ in his brake-pedal pressure from skid to skid.
3. The driver may miss the test speed by a considerable amount.
4. The "pull-apart" connector for the chalk marker did not release at the same instant for each brake application. This error would not necessarily be corrected by connecting the marker to the brake-light system as this system has large variations in "tripping" pressure.
5. The considerable time required to measure each skid with a tape created serious traffic problems and evaporation often made it impossible to run more than one skid at a site on hot days without rewetting. Wetting a surface before each skid would impose a problem in the procurement of sufficient water and require much more testing time.

It was thus decided to modify the test car to minimize or eliminate these disadvantages. The idea evolved to outfit the car with some type of electrically operated power brake that would give a constant braking pressure with each application. This could be connected with an accurate fifth-wheel speedometer-odometer that would automatically close the braking circuit at a predetermined test speed and then measure the distance required to stop.

Letters explaining the problem were sent to many major concerns dealing with the manufacture of power brakes and speedometers. It soon became apparent that a speedometer that would close a circuit at a predetermined speed would be difficult to construct, and the necessary centrifugal switch would have a rather wide range of closing values. An alternative electrical speedometer could do the job accurately, but proved too expensive and would not measure distance. It was therefore decided to close the brake circuit manually and to find an instrument that would record the speed as the brakes were applied. The "Wagner Stopmeter" manufactured by the Wagner Electric Company of St Louis, Missouri, fulfilled this requirement and was available at a reasonable cost. A unit was subsequently purchased.

A study of possible braking systems revealed that a vacuum brake system would be the most practical and economical for test purposes. Air pressure and electrical brakes were investigated, but the cost of adapting either to the test car would have been far greater than that for the vacuum system. The Bendix Products Division of the Bendix Aviation Corporation of South Bend, Indiana, proposed a very simple, electrically-operated, vacuum-braking unit. This unit was eventually constructed and installed in the test car by the Bendix Corporation.

The Bendix System is illustrated in Figure 1. The heart of the system is the vacuum unit that operates a Ford master cylinder (E) through a simple lever system (F). The vacuum chamber (A) is activated when an electrical impulse opens the solenoid valve (D) which, in turn, opens the vacuum valve (C).

The vacuum thus created in the left side of the vacuum chamber causes atmospheric pressure to force the lever system (F) to the left, thus activating the master cylinder (E). The vacuum is supplied by the intake manifold of the engine through a line (J). A one-way valve (I) was installed to eliminate any losses in the vacuum reserve tank (B) during periods of low manifold vacuum. A brake fluid reservoir (H) was included with lines running to master cylinder (E) and to the car master cylinder, so as to eliminate the possibility of pumping fluid between the car system and the power system.

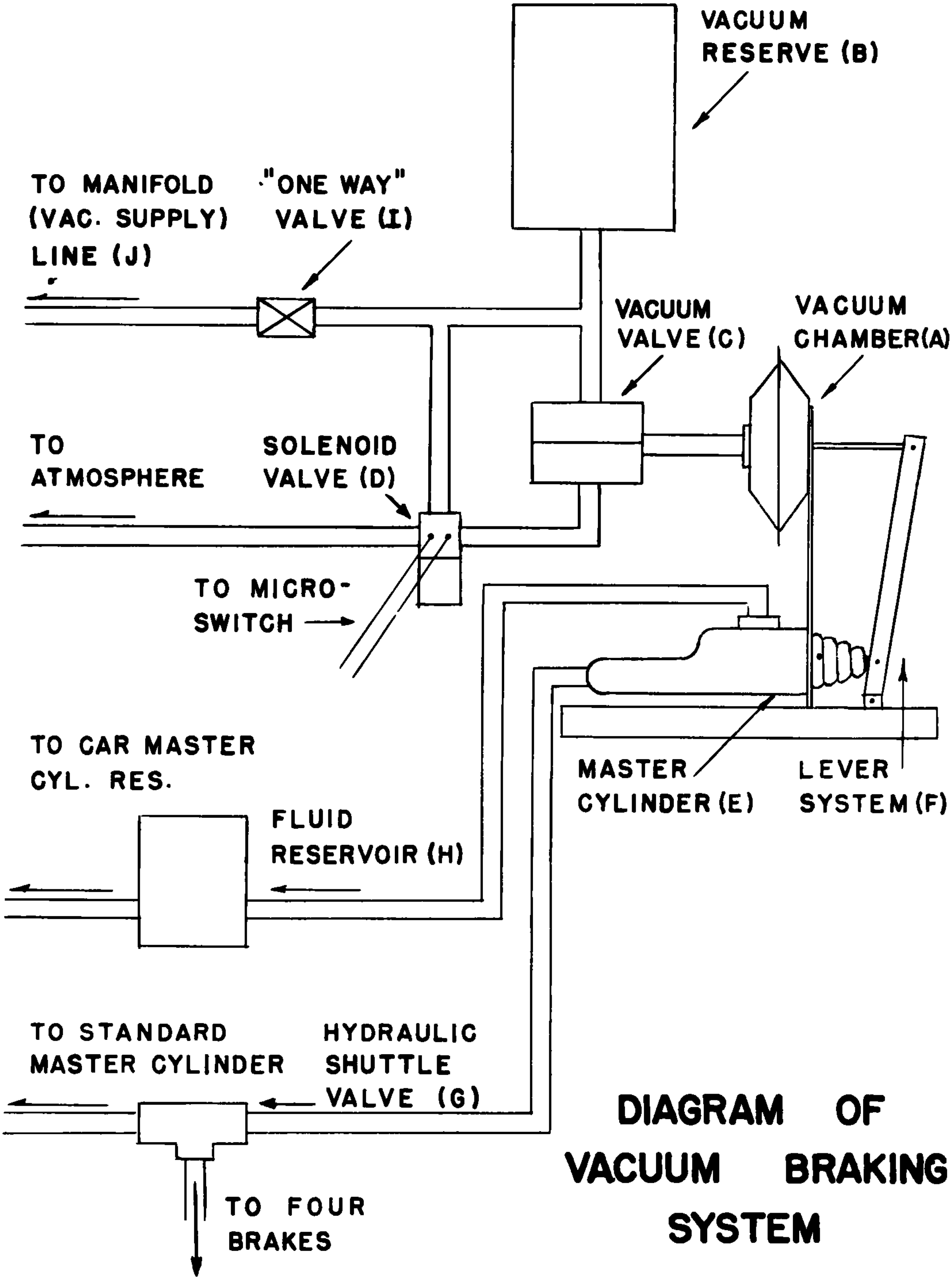

Figure 1.

Line pressure applied by master cylinder (E) causes the shuttle valve (G) to close the line to the standard car master cylinder and allows line pressure to be distributed to the individual wheels for braking. When the current to the solenoid valve (D) is discontinued the pressure is instantly relieved and application of the standard car brake will transfer the shuttle valve (G) and allow normal operation of the brakes. The diameter of the shuttle valve was reduced from $\frac{1}{2}$ in. to $\frac{1}{4}$ in. to eliminate the necessity of a large amount of fluid displacement when transferring from this special braking system to the normal method. The system, as outlined, was installed in the trunk of the test car by Bendix personnel and operated without incident through the entire series of tests. The power system applies a brake line pressure of approximately 700 psi and locks the wheels in less than 0. 17 of a second.

With the braking problem solved, there remained a need for a method which would permit the driver to conveniently lock the brakes and activate the Wagner Stopmeter at the desired speed. After consideration of several types of foot switches, it was apparent that more positive and sensitive driver control could be realized by a hand-operated switch. A micro-switch was mounted on the steering wheel rim and connected to a circular copper contact plate at the wheel base. A carbon brush was set in the steering column so that it was in contact with the circular plate for all positions of the steering wheel and was connected into the coil of a 6-volt double-pole relay. The relay was connected as shown in the wiring diagram (Figure 3) to the solenoid valve and the odometer and speedometer.

These modifications made skid testing quite simple and minimized the driver variable. In order to make a test run the driver merely has to let the car slow down to the test speed as indicated by the special speedometer and press the micro-switch. This action locks the brakes and at the same instant it holds the speedometer and activates the odometer. At the end of the skid it is possible to record the braking speed and the skid distance from the stopmeter dials. This testing method has proven to be extremely consistent.

The preliminary tests indicated a problem with the brake-backing plates. The manufacturer, however, supplied the project with special reinforced plates and no further difficulty of this type was encountered.

Method of Testing

Each selected road is tested in at least three locations, with two skids performed at

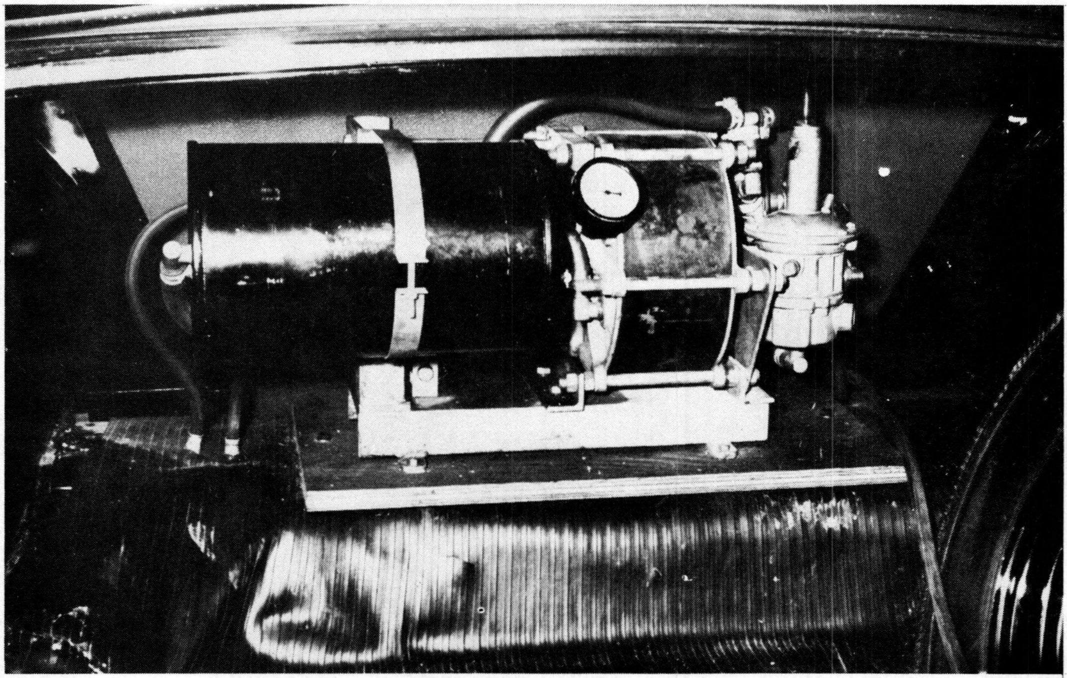

Figure 2. The Special Bendix Vacuum Braking Unit was installed in the trunk of the test vehicle.

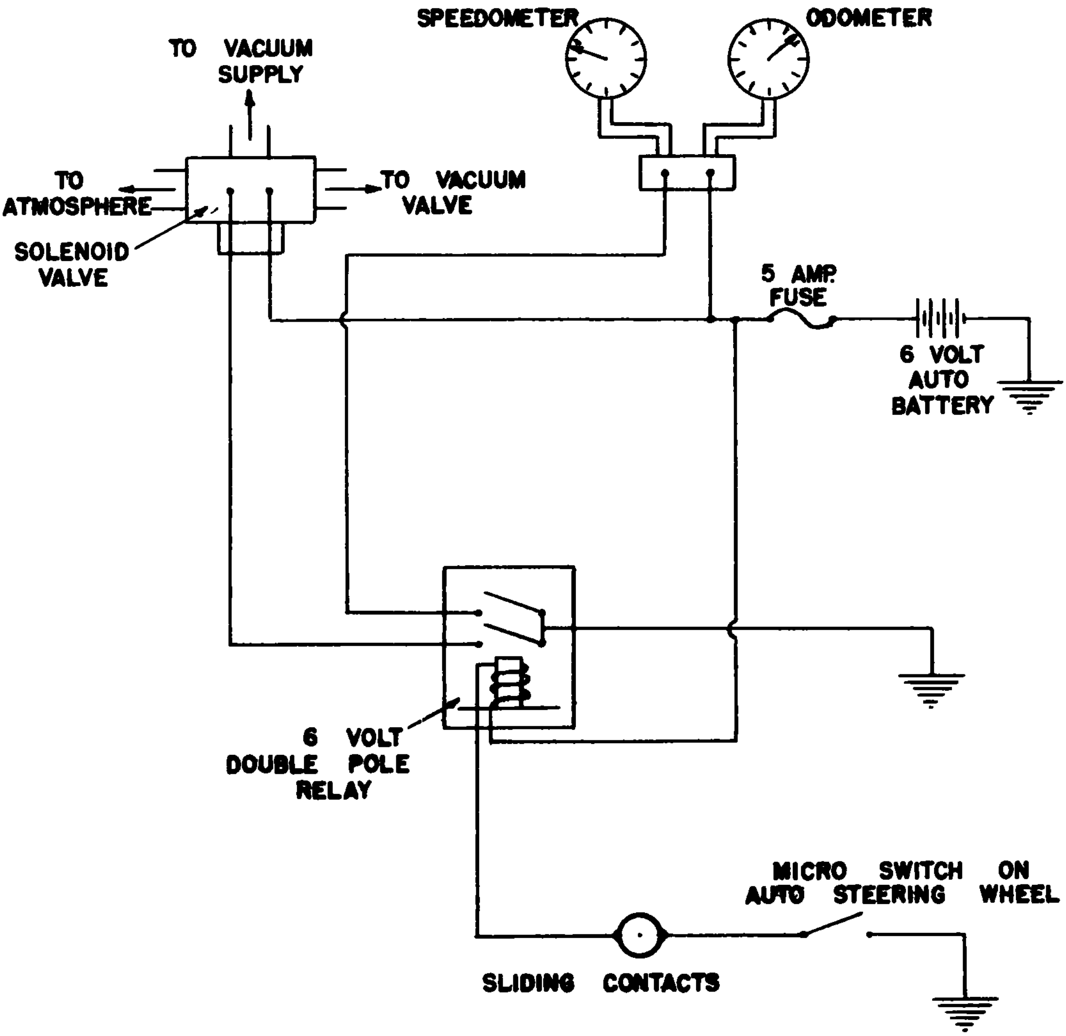

Figure 3. Wiring diagram of braking circuit.

each location. A location is a level, straight stretch of pavement 200 to 300 feet in length and located anywhere on the road. It is usually possible to select the locations in such a manner as to allow adequate sight distances for flagmen to stop traffic safely.

Most skids are run at 30 mph. The primary reason that one speed was chosen was that the interest is in comparing the skid resistance of the various road surfaces and not in studying the effects of various speeds. The selection of one speed also allows more roads to be tested in the available time. Thirty mhp was selected as it was considered to be the highest speed that could safely be used over the wide range of road types that exist in Indiana. The fact that the test car left the road many times during the testing to date indicates that a greater speed would be quite hazardous.

The purpose of these tests is to investigate the skidding properties of roads and since the wet condition is the most critical, almost all of the tests are run under wet conditions. The only equipment necessary is the test vehicle, a water truck, two red flags, two thermometers, data sheets, clipboard and pencil. The usual crew consists of four men; the test car driver, an assistant, a water truck driver and a flagman. The four man crew is sufficient for most roads, but an additional flagman is desirable on highly congested routes.

After a location is selected on a particular road, the test car and flagman stop at a point 400 to 500 feet in advance of the test section and the flagman halts all traffic approaching from the rear. The water truck and test assistant proceed ahead to the test site and begin wetting without interferring with oncoming traffic. The test assistant controls the water from the truck. When a sufficient strip (200 to 300 feet) is thoroughly wetted the assistant remains at the site and the water truck driver pulls the truck

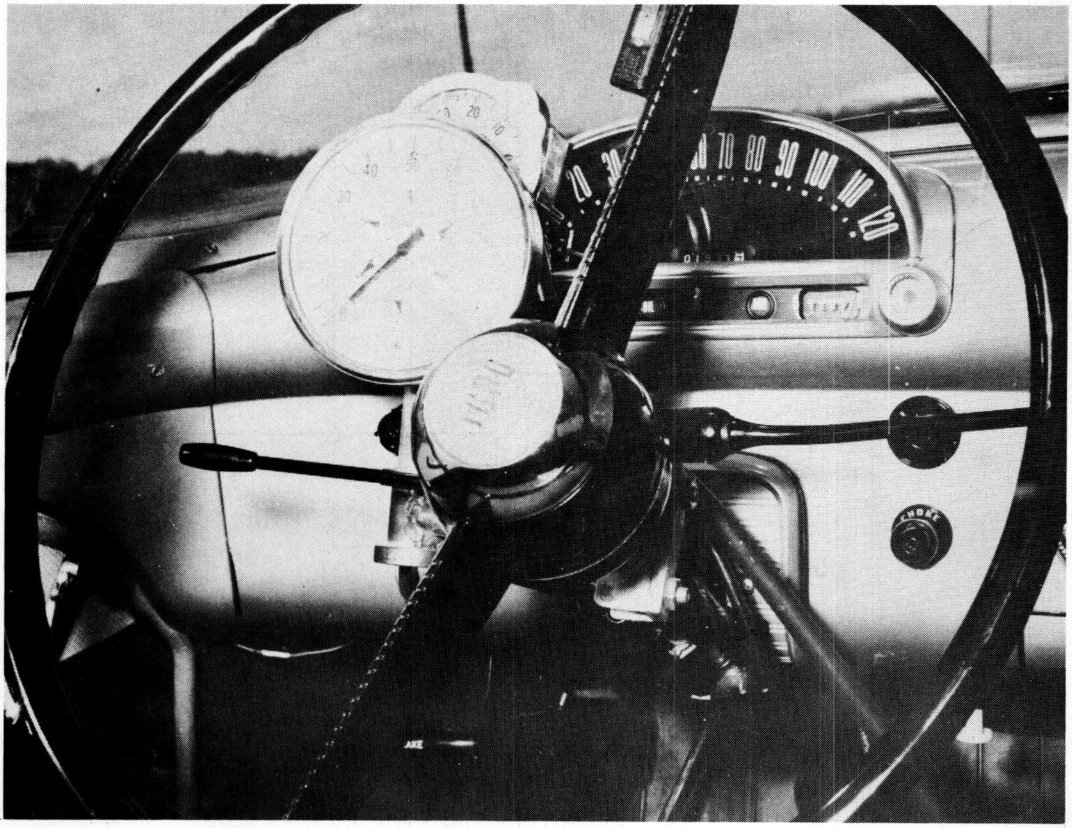

Figure 4. The speedometer and odometer dials of the stopmeter were mounted inside the skid vehicle.

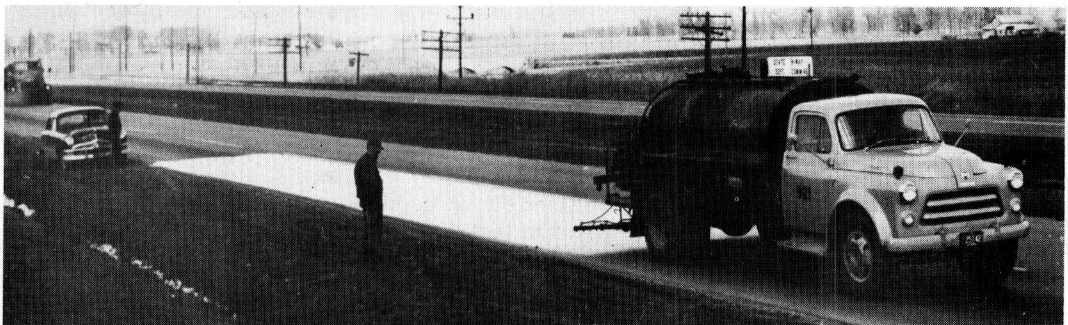

Figure 5. The pavement was thoroughly wetted before each series of skids.

Figure 6. After the pavement was wetted, the skids were performed.

several hundred feet ahead, dismounts and stops oncoming traffic. The two skids are then run in quick succession, with the assistant and flagman raising and lowering the fifth-wheel as necessary, as it is not possible to back the car with this wheel down. It is generally possible to run both skids in less than two minutes after completion of wetting thus keeping the effects of evaporation and runoff to a minimum. The speed to the nearest 0.25 mph and distance to the nearest 0.25 foot are read from the stopmeter dials and recorded by the driver after each skid.

THE PRESENT PROGRAM OF TESTING

The skid project in Indiana developed along several lines. Two hundred thirty-three roads were tested during the summer of 1954. The roads tested were selected as a sample of the state highways in every part of Indiana and included roads of various surface types, different volumes of traffic, and various ages. The data obtained during this program have been analyzed and the results are indicated later in this paper.

Indiana also has several experimental pavements and on some of these periodic skid tests are continuing. An annual skid test is performed on a surface containing silica sand to determine the long-term skid characteristics of this surface type. Semi-annual skid tests, one in the summer and the other in the winter, are also conducted on the US 31 Test Road. A section of this road near Columbus, Indiana, is constructed for a long-term comparison of the characteristics of portland cement concrete and bituminous concrete. One of the characteristics being compared is that of skid resistance.

Another use of the skid equipment is in evaluating reported "slick" sections of highway. As the state police or highway department receives complaints of "slick" highways, the location is referred to Purdue and a skid test is scheduled for the reported location. A confirmation of the hazard by the test initiates activity by the responsible agency to eliminate the hazard.

DISCUSSION OF RESULTS FROM STATEWIDE STUDY

Four major construction types were investigated: rock asphalt, portland cement concrete, bituminous concrete and other bituminous surfaces. A total of 233 different roads were tested in a wet condition, and 20 in the dry condition. Each road was tested at three locations, and

Figure 7. The Wagner Stopmeter was attached to the rear of the skid vehicle.

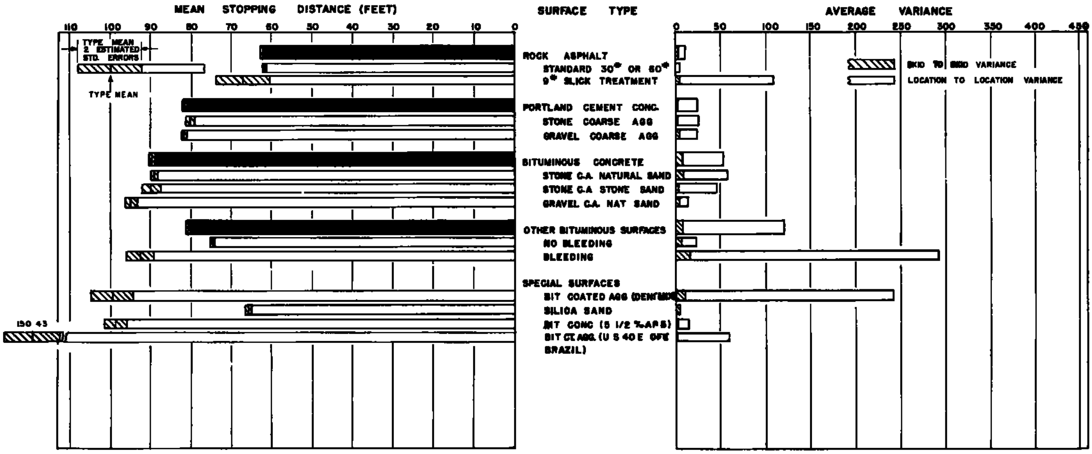

Figure 8. Estimated means and variances for the survey roads.

two skids were taken at each location. All tests were made at a brake-application speed of 30 mph. All distances indicated in this report are average stopping distances from the instant of brake application and were obtained using the same skid vehicle.

For each selected location, any difference in the two skid distances gives rise to a variation which is called the skid-to-skid variance.

The three selected locations for a particular road always have somewhat different mean skid distances, and thus there is also a variation that arises from these differences. The measure of this variation is called the location-to-location variance.

Whenever more than one road has been selected in a given type or subtype, differences among the observed road means give rise to a third variance, the road-to-road variability.

TABLE 1

SUMMARY OF MEAN SKID DISTANCES, AVERAGE VARIANCES, AND STANDARD ERRORS

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
CODE NO	SURFACE TYPE	MEAN SKID DISTANCE (FEET)	NO OF ROADS	AVERAGE VARIANCES SKID TO SKID	LOCATION TO LOCATION	ROAD TO ROAD	STD ERROR OF THE MEAN * (FT.)	STD. ERROR OF THE MEAN ** (FT)
IXX	ROCK ASPHALT	62 56	32	1 02	11 39	34 23	0.24	043
111	STANDARD 30* OR 60*	62 25	30	0.84	4.98	25.51	0 17	0.36
121	9* SLICK TREATMENT	67 27	2	3 84	107.53	38 17	2 98	1 78
2XX	PORTLAND CEMENT CONCRETE	81.44	46	1.87	23.83	422.36	0.29	1 24
211	STONE COARSE AGG.	80 78	17	1 32	25 82	397 39	0 50	1 97
221	GRAVEL COARSE AGG.	81 83	29	2.06	22 67	446.28	0.36	1 60
3XX	BITUMINOUS CONCRETE	89.22	59	6 79	53.00	827.45	0 39	1.53
31X	STONE COARSE AGG.	88 78	55	6.99	55.65	830.02	0.41	1 59
311	NATURAL SAND	88.64	48	7.62	57 14	923.30	0 44	1 79
312	STONE SAND	89 80	7	2 64	45 43	199 99	1 04	2 18
321	GRAVEL C.A.-NAT. SAND	95 21	4	3.99	13.12	82 43	0.74	1 8ᴿ
4XX	OTHER BITUMINOUS SURFACES	80.83	47	8.20	119.97	1,304.50	0.21	2 15
41X	NO BLEEDING	74 06	30	5.11	22 34	508.30	0.36	1 68
411	ALL GRAVEL	77 86	12	6 43	19.10	618.22	0.51	2 97
412	ALL STONE	71 50	18	4.24	24 51	353.57	0 48	1 81
42X	BLEEDING	92 77	17	13 84	292.26	1,428.55	1 69	3.74
421	ALL GRAVEL	103 44	8	23.71	355.32	1,037.55	2 72	4.65
422	ALL STONE	83.28	9	4 69	236 41	672.01	2.09	3.53
5XX	SPECIAL SURFACES	—	14	—	—	—		
51X	BIT CTD. AGG. (DENSE MIX.)	99.38	6	8.03	200.85	918 46	2 36	5 05
511	ALL GRAVEL	103.01	3	9 94	174.70	116 77	3 12	2 55
512	ALL STONE	95.76	3	6 12	226.99	1,942.84	3 55	10 39
52X	SILICA SAND	66.68	1	2 82	0 83	—	0.37	—
53X	BIT. CONC. (5 1/2 % AP 5)	98.35	1	1.85	13.74	—	1 52	—
54X	BIT. CT AGG. (US 40 — 4 MI. EAST OF BRAZIL)	150 46	1	0.24	59.30	—	3.14	—

* BASED ON ONLY THOSE ROADS INCLUDED IN SURVEY

** REGARDING THE SELECTED ROADS OF EACH TYPE AS A RANDOM SAMPLE OF INDIANA ROADS

Skid-to-skid variances are generally the smallest of the three types of variability since the two skids taken at the same location represent tests under the most similar conditions. The location-to-location variances represent differences that arise from variation in the surface properties of a particular road, from place to place, probably due to such factors as construction differences, varying traffic, and bleeding. Road-to-road variances, within a given type or subtype, reflect differences in skid resistance that must be associated, for the most part, with discrepancies in age or wearing, materials and methods of construction, and volume and type of traffic to which the roads have been subjected.

The results of the tests for each road type are discussed in the following sections and are summarized in Table 1 and Figure 8. All means stated are for a brake-application speed of 30 mph with the road surface in the wet condition. Significant differences are said to exist only when two-standard-error "regions of uncertainty" do not overlap. The standard errors used are based on only those roads included in the survey. The three variances, as previously discussed, are also shown in Table 1 and Figure 8.

It must also be noted that the difference in skidding distance between two roads or road types is given in this study for a speed of 30 mph. A much greater difference in distance would be obtained at higher speeds and could be a very critical element, certainly a very dangerous one.

Rock Asphalt

The rock asphalt roads tested displayed excellent skidding properties in the wet condition. The mean of 180 skids on 30 different roads at 30 mph was 63. 3 feet, a value significantly lower than that for any other major type, and only a few feet greater than the mean of the roads that were tested in the dry condition. The variability of the skid distances found on these roads was also found to be of an especially low order, the road-to-road variance being on a par with the skid-to-skid variances for many other types and sub-types.

An attempt was made by rank order correlation and graphing to find a relationship between the small amount of variation existing between these roads and traffic volume, but no such relationship appears to exist. The skidding characteristics of rock asphalt surfacing are, then, not apparently affected by traffic volume and age; surfaces 14 and 15 years old were found to have about the same mean skid distance as many of those of recent construction.

Two roads that had been "deslicked" by a thin 9 lb rock asphalt treatment were included in the survey. One of these roads was still completely covered, while a considerable portion of the resurfacing had worn off of the second. The former road displayed a mean and variance very similar to the conventional rock asphalt roads tested, while the latter had an exceedingly high mean skid distance and variance. It was concluded that the 9 lb treatment is an effective method of temporarily "deslicking" surfaces. The type and condition of the previous surface is probably an important factor governing the service of the "deslicking" treatments.

These tests on rock asphalt surfaces serve to substantiate the conclusions of many other experimentors that those surfaces with a harsh, gritty, sandpaper finish have superior skidding properties in the wet condition. The only other surfaces tested that had similar properties were two sections constructed with silica sand, and they, too, had excellent skidding properties and small variances.

Portland Cement Concrete

The over-all mean of 276 skids run on 46 roads constructed of portland cement concrete was 81. 4 feet. This value is significantly higher than that for rock asphalt, but significantly below the estimated mean for bituminous concrete.

Although the average skid-to-skid and location-to-location variance for these roads were over twice as high as those for rock asphalt, and the road-to-road variance was over 10 times as great, these variances were seldom more than half of those of the other major road types.

The comparatively small amount of variability on these roads is especially significant in view of the fact that these roads average to be considerably older and have carried more traffic than any of other major types, indicating that although the roads tested varied tremendously in both age and traffic volume, there was comparatively little variation in their skidding properties. An attempt was made to correlate the mean skid distances of all the tested roads with average traffic volume, age, and total traffic but no significant relationships appeared to exist. The roads were then separated into two groups: pre-1945 and post-1945. Both of these groups were studied individually for a relationship between mean skid distance and both average volume and total traffic volume (the product of age and daily traffic). The pre-1945 group indicated that there is no relationship between skid resistance and traffic for these older roads and thus their variability must arise from other factors not included in this investigation. The post-1945 surfaces indicated a definite increase in mean skid distance with increases in both average daily traffic and total traffic. Another conclusion indicated by this study is that traffic tends to decrease the skid resistance of portland cement concrete surfaces to a measurable degree for a few years after construction until they reach a point beyond which "polishing" action is greatly retarded. This premise was further substantiated by the fact that the post-1945 roads tested yielded a mean skid distance approximately 5 feet shorter than that for the older roads.

The portland cement concrete roads tested in the survey were also divided according to the type of coarse aggregate (gravel or stone) used in the mix, but no over-all differences in either means or variances were indicated between the two types.

Bituminous Concrete

The mean skid distance found for the 59 bituminous concrete roads tested was 89.2 feet at 30 mph. This mean is significantly greater than that for those roads tested in any other major type. These surfaces were also quite variable, yielding variances almost twice those for portland cement concrete for each of the three sources of variation.

The bituminous concrete roads were divided into three major groups for comparison purposes: those constructed with stone coarse aggregate and stone sand, those with stone coarse aggregate and natural sand, and those constructed with gravel coarse aggregate and natural sand. Contrasts among these groups indicate that, for the roads tested, the roads containing stone coarse aggregate have a significantly lower mean skid distance than those constructed with gravel. Another contrast indicated that there was no significant difference between the roads constructed with natural sand and those containing stone sand. It is interesting to note that although the gravel coarse aggregate roads had the highest mean skid distance among the three sub-types studied here, the location-to-location and road-to-road variance is of an exceptionally low order, revealing consistency among these surfaces. It should also be pointed out that the road-to-road variances found for the stone sand roads were also extremely small, being less than half those for portland cement concrete, again suggesting a relatively consistent type of surface.

The effects of traffic on the three types of bituminous concrete were studied separately. The stone coarse aggregate, natural-sand fine aggregate roads gave indications of increasing mean skid distance with increasing total volume. The information on the other sub-types indicated no effect due to traffic volume.

Other Bituminous Surfaces

All the bituminous surfaces tested other than rock asphalt and bituminous concrete have been grouped together under the general heading of other bituminous surfaces. These bituminous surfaces represent a considerable percentage of the total highway mileage in the state.

The 55 bituminous surfaces considered here yielded a comparatively low over-all mean of 80.8 feet; a value almost identical to that of the portland cement concrete roads, and significantly lower than that for the bituminous concrete surfaces tested. The variability among these roads was, however, of a large magnitude, far exceeding that of any other major type for each of the three sources of variation. Individual skids on these roads ranged from 50 to 167 feet, displaying skidding properties anywhere from excellent to very poor. These high variances make it possible for any road or location to yield an average skid resistance value that is radically different from the comparatively low indicated mean.

For initial comparison purposes, the bituminous surfaces were divided into two groups, those surfaces containing stone aggregate, and those containing gravel. Those road surfaces containing gravel were found to have a significantly higher mean than those surfaced with stone. The gravel roads also displayed a greater road-to-road variance than stone, although the site-to-site and location-to-location variances did not differ to any great extent.

The inconsistent nature of these roads stems, to some extent, from the many variables inherent in these surfaces and from the many different combinations of aggregate and bituminous material that are present. The skidding properties are also influenced by the previous construction history of any particular road.

The major cause of variability for these roads, however, appears to result from the bleeding of excess bituminous material, either from the current or previous construction. This bleeding causes "fat spots" to appear, usually along the wheel tracks, but often over the entire road. These bleeding sections made it necessary for friction to be developed primarily between the bituminous material and the tire, as the aggregate was usually wholly or partially buried. High mean skid distances and high location-to-location variances were almost invariably associated with bleeding. Several bleeding roads were found to be too hazardous to test as it was impossible to keep the test vehicle on the road for the full length of skid.

In order to evaluate the effects of bleeding and to make further comparisons, these bituminous surfaces were again divided into two major groups: those that evidenced some bleeding and those that were entirely free from bleeding. Comparisons were then made both within and between the groups.

The non-bleeding surfaces, as a group, displayed a mean of 74. 1 feet, a value considerably below that for any major road type other than rock asphalt. The variances, too, were of a reasonably low order, especially the average location-to-location variance which was very close to the value for portland cement concrete.

The group of bleeding surfaces, on the other hand, was found to have the relatively high mean of 92. 8 feet, a figure significantly higher than that for the non-bleeding surfaces, exceeding this value by over 18 feet. The skid-to-skid variance was somewhat greater for the bleeding roads and the road-to-road variance almost three times as great, but a rather spectacular contrast was found in the location-to-location variances. The location-to-location variance for the bleeding surfaces was over ten times that for the non-bleeding ones. This indicates that a considerable portion of the variability and high means in these surfaces can be explained by bleeding. It may also be seen that these bituminous surfaces can have very good, consistent skidding properties if no bleeding occurs.

Comparisons among aggregate types, sizes, and bituminous materials were made within each of the two groups. The gravel roads were found to have a significantly higher mean within both the bleeding and non-bleeding groups, although the difference was much more pronounced for the bleeding group. This contrast further indicates that stone has better initial skidding properties even if no bleeding occurs. The variances within both groups were quite homogeneous, and neither aggregate appears to give more consistent surface than the others.

Contrasts among the three prevalent sizes of aggregate in each group did not reveal any notable or significant differences in means or variances from size to size for either gravel or stone. Thus, for these roads, aggregate size does not appear to affect the skid distance to any measurable degree.

Special Sites

Several surfaces were tested during the summer program that do not strictly fall into any of the previously discussed classes, and, therefore, merit individual consideration.

The first of these is a silica sand surface on US 46 east of Greensburg. This road has an appearance and texture very similar to that of rock asphalt. The mean skid distance on this section was 66. 7 feet, which was significantly lower than any road type except rock asphalt. The variability, especially that from location-to-location, is of an especially low value, indicating consistency similar to rock asphalt.

A group of six bituminous coated aggregate surfaces was also included in the survey. These are designated as "dense mix" and had a plant mix surface composed of No. 14 sand, No. 11 gravel or stone aggregate combined with either RC 5 or AE 90. The overall mean of these six roads was a comparatively high 99.4 feet and both the site-to-site and location-to-location variances were quite high. This mean was significantly higher than that for any of the major road types tested.

One surface tested clearly illustrated the seriousness of skidding due to polishing of aggregate. On a section of US 40 near Brazil a bituminous coated aggregate surface has been exposed to heavy traffic throughout its entire life. No bleeding was evident on the road and much of the seal had worn off leaving a very coarse looking "open" surface. The mean skid distance on this road was 150.5 feet — the highest of any road tested in the survey. Close observation revealed each stone to be rounded and highly polished. Comparatively low variances on this road indicate this road was consistently slick from place to place.

CONCLUSIONS

The conclusions of this study are:

1. The vehicle stopping-distance method that utilizes an electrically controlled vacuum braking system and an integrated fifth wheel speedometer and odometer produces consistent and reproducible results.
2. The vehicle stopping-distance method is economical, rapid, and relatively safe at 30 mph.
3. Of all surfaces tested, rock asphalt has the best skid characteristics when wet, both as to average distance and variability.
4. A thin application of rock asphalt is a good but temporary method of deslicking a pavement.
5. Portland cement concrete surfaces provide relatively good skid characteristics but are subject to polishing by traffic during the first few years of their life.
6. Bituminous concrete surfaces, as constructed in Indiana under present specifications, do not have as good skid characteristics as rock asphalt or portland cement concrete.
7. Bituminous surfaces other than rock asphalt and bituminous concrete have a relatively low average skid distance but are very variable.
8. Bleeding on bituminous surfaces results in a significant increase in the stopping distance.
9. Bituminous surfaces constructed with Indiana limestone aggregate exhibited better skidding characteristics than those constructed with Indiana gravel aggregate.
10. Bituminous surfaces that were coarse and open exhibited poor skidding characteristics.
11. Although it cannot be considered conclusive because of the small number of roads tested, surfaces constructed with silica sand gave good results, comparable to rock asphalt.
12. Since bituminous surfaces exhibited skidding distances when wet ranging from 50 to 167 feet, excellent to very poor, it is apparent that they can be designed and constructed so as to give excellent skidding characteristics.

ACKNOWLEDGMENTS

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Finally, thanks is expressed to Dan Kleehammer, test assistant, who provided assistance and morale during the long, eleven week field survey.

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Discussion

P. C. SKEELS, Head, Experimental Engineering Department, K. A. STONEX, Head, Technical Data Department, General Motors Proving Ground, and E. A. FINNEY, Research Engineer, Michigan State Highway Department — Engineers in the automotive industry are as much concerned as the highway engineers in the problem of maintaining high and consistent road surface friction characteristics on highways. Nothing is so important for the safe, effective use of the highway system, and a joint effort to determine the reasons for poor friction characteristics and to develop solutions is a matter of paramount importance.

This discussion is concerned primarily with the decrease in wet friction characteristics which results from a polishing action of the components of the pavement surface as a fundamental cause. The problem related to contamination of the surface by traffic slick or other factors is considered supplementary, however important it may be. The discussion is essentially as contained in a paper* by the writers presented at a meeting of the Association of Asphalt Paving Technologists.

It is evident to the eye and by feel that the aggregate in pavement surfaces does develop a high degree of polish in many instances. High-speed motion pictures showing a tire running over a piece of tempered glass make it clear that there is an appreciable movement of the portions of the tire as each section rolls through the contact area. The total movement of the ribs increases from the center to the outside; on the particular tire observed the contact area narrowed $\frac{5}{16}$ inch, which means that each outer rib moved approximately $\frac{5}{32}$ inch toward the center and back. Compression in the longitudinal direction causes a movement of as much as $\frac{1}{8}$ inch.

There are other components in this complex motion. It should be pointed out that this configuration is that of a free rolling front tire and that tires under braking, or rear tires driving, will have other slight longitudinal motions at the trailing edge. This continuous scrubbing action repeated thousands of times a day upon a heavily traveled road must inevitably wear away the pavement surface, and produce a high degree of polish on pavement surface components susceptible to polishing.

This relative motion between the tire and the road will also wear off tread rubber. This has lead the tire manufacturers to devote a large amount of development work to designing tread patterns and contact shapes that will minimize total wear and distribute the wear evenly across the tread. It must be concluded, therefore, that this wiping action is an inherent characteristic of an annular tire deflected by reaction with the road surface, and that everything possible is being and will continue to be done to minimize the amount of this polishing action. The most hopeful steps for significant progress seem to lie in the area of developing pavement surfaces which do not become polished under abrasive wear.

Test observations give a strong basis for the assumption that both portland cement and bituminous concrete surfaces become increasingly slippery when wet as total traffic and wear accumulate, and that this deterioration in friction capacity is related to the degree of polish of the components of the surface.

Because of the deep interest of the automobile manufacturers in the whole highway program, the General Motors Proving Ground Section was pleased at the opportunity to cooperate with the Michigan State Highway Department in performing the brief tests discussed here. The writers' part in the cooperative test was to develop inexpensive and simple instrumentation which could be used at an early date to survey selected road surfaces without interrupting normal traffic flow.

The basis of the design is that rear wheel torque reactions developed by driving or braking forces on a car equipped with a torque tube drive are transmitted through the torque tube and produce a deflection proportional to the wheel reaction (Figure 1). A 1954 Buick Special was selected for the test, and wire resistance strain gauges were cemented to the torque tube in such a manner that deflection of the tube would be indicated by appropriate strain measuring instruments. In this application a Servo-type indicator, developed and constructed by the Proving Ground, was used. However, any other type of strain indicator could be used equally as well. The same set of production tires was used throughout the test program; these tires were thoroughly broken in, but the treads were in good condition.

The hydraulic brake lines to the front wheels were fitted with valves, so that the front brakes could be blocked out and the braking reaction be developed by the rear wheels only. To avoid the hazards of sliding stops in traffic and to simplify the problem by making measurements at constant speed, the test car was towed behind a truck.

A tank truck fitted with a valve so that water could be sprayed on the road just ahead

*Skeels, P. C., Stonex, K. A., and Finney, E. A., "Road Surface Friction from the Standpoint of Automotive and Highway Engineers." Presented at a meeting of the Association of Asphalt Paving Technologists, Cleveland, Ohio, Feb. 13-15, 1956.

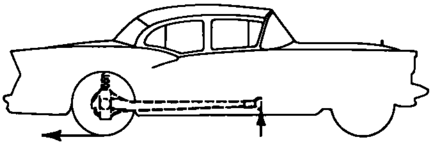

Figure 1. Schematic diagram illustrating bending moment in torque tube.

of the test car provided a source of water to wet the surface.

The outstanding advantage of this instrumentation was that it used existing components which were adapted to this survey with comparatively little effort. The chief advantage from an operating standpoint is that the speed of the vehicle train is not changed as the test observation is made, this uniformity of speed permitting safe test performance at normal traffic speed without disturbance of the traffic stream and yielding test results which are not influenced by possible changes in coefficient of friction with speed.

A further advantage from the point of view of a quick survey is that the use of an indicating type of meter gave immediate results, so that the test program was flexible and could be adjusted in the light of the continuing flow of results.

However, a comprehensive program on a continuing basis might be served better by the use of a trailer fitted with a controllable braking system and probably by the use of recording rather than indicating instrumentation.

The test procedure consisted of towing the car along the roadway at a speed of 40 mph; at the desired test site, the water was turned on to flood the pavement.

The driver applied the brakes slowly up to the point of skid, the car was dragged momentarily with the rear wheels sliding, and then the brakes were released and the water was stopped until the next test observation was made. In the meantime, the test observer watched the strain indicator as the brake reaction rose to the maximum at the point of incipient slide and then fell away abruptly to a more or less stable point during the slide. The incipient sliding value and the sliding value were both noted. Repeat tests were made throughout the length of any given project to average out local irregularities and variations of coefficient of friction; on occasions the train was disconnected and the vehicle brought back to the beginning so that check runs could be made over the same course. The consistency of test results under similar circumstances led to the belief that the spray of water was sufficient to flood the paved surface completely and to give reproducible test conditions.

Immediately prior to the tests on the Michigan highway surfaces, a heavy rain had fallen; the pavement surface appeared to be clean, and there was a reasonable certainty that the surfaces tested were as free from contamination of traffic slick, oil drippings, and dirt as is ever apt to be found. Therefore, the results are considered applicable to surfaces as clean as found normally.

Prior to the test observations, the strain indicator was calibrated by locking the brakes with the test car stationary and observing the relative readings of the strain indicator and a traction dynamometer in the tow cable under an extended range of values.

Since the tow cable is attached some distance above the ground, the force in the tow cable and the rear wheel friction force produce a couple which is balanced by an effective transfer of weight from the rear to the front wheels. The weight transfer is computed from the height of the tow cable and the wheel base of the car, and the coefficient of friction determined from the indicated reactions is adjusted to take into account this transfer of weight. The coefficient of friction is defined here as the ratio of the horizontal force and the vertical reaction at the rear wheel. Derivation of the formula is given in the Appendix. At increased speeds the reaction developed by the air resistance couple may become significant.

The strain indicating meter was mounted in the front seat compartment of the car and a portable radio was used for communication with the tow truck. Prior to the formal test program, evaluation tests were made on parts of the Proving Ground road system.

Figure 2 shows the incipient and sliding coefficients of friction observed in the 40-mph lane of the Proving Ground test track with the pavement as described before. With one exception, these results are consistent within a range of coefficient of friction of ± 0.01 and the incipient slide is consistent within a range of coefficient of friction approximately ± 0.02. The accuracy with which the strain indicator could be read is not much better than this.

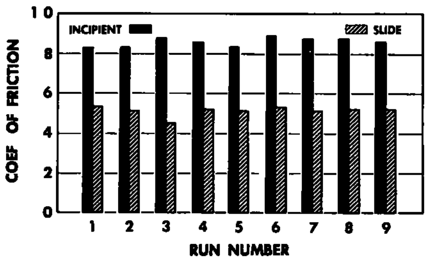

Figure 2. Incipient and sliding coefficient of friction observed on the 40-mph lane of the Proving Ground test track (bituminous concrete surface).

The test track surface is a bituminous concrete material corresponding to Michigan highway Class 1 with 25A coarse aggregate specification. At the time of this test, the surface had been in use for approximately two years under extremely light traffic volume by normal highway standards.

Comparable observations at a speed of 50 mph show that the effect of speed is not significant statistically.

The uniformity of test results on another bituminous concrete used on the Military Proving Ground straightaway is not quite as close, possibly because the surface variation may be somewhat greater. This surface was about four years old at the time of the test and had been subjected to a very light volume of military vehicles, both wheeled and track laying.

Consistency of observations made on a portland cement concrete engineering test straightaway, which has had a very light traffic volume during a period of about eight years service, is within the accuracy of observation of the strain indicator, and the mean values of a sliding coefficient appear to be measurably higher than those of the bituminous concrete surface.

Other tests were made on a sheet asphalt surface with extremely low traffic volumes and of about six years service, and on a public highway adjacent to the Proving Ground and surfaced with an oil aggregate type surface with about two years service under light traffic. Also recorded were dry friction measurements on both the bituminous and portland cement concrete surfaces.

The areas tested on the Michigan highway system were selected earlier by a visual survey. These test situations included variations in aggregate type, traffic volume, and length of service. Although they were confined to bituminous concrete surfaces in the original objectives of the survey, several portland cement surfaces were included for comparative purposes.

By plotting the coefficient of friction observed against a wear factor determined by multiplying the average daily traffic volume per traffic lane over the period since construction by the length of service in years and dividing by 1,000 to express the index in convenient numerals, the results for bituminous concrete with limestone aggregate, bituminous concrete with gravel aggregate, portland cement surface, and the composite of the three, there is a strong indication that the three basic types of surface give distinct wear index behavior patterns.

A study of certain apparent abnormalities in the bituminous surface plottings has uncovered facts believed to be significant to the problem of constructing highway surfaces which will maintain a high degree of skid resistance throughout their useful life. For instance, projects constructed prior to 1944 involve material specifications different in certain respects from those of the balance of the projects, which were constructed since 1944. The differences in material specifications are described as follows:

	Projects Prior to 1944 3 - 5 - 7 - 20	Projects 1944 - 1948 8 - 9 - 14 - 15 - 18	Projects Since 1948 Balance Studied
Asphalt cement	Pen. 85-100	Pen. 85-100	Pen. 60-70
Mineral filler	Limestone dust	Limestone dust	Flyash
Coarse aggregate	100% passing ½-in. sieve 15-45% passing No. 4: Dept Spec 26A	90-100% passing ½-in. sieve 0-25% passing No. 4: Dept Spec 26A mod	90-100% passing ½-in. sieve 0-25% passing No. 4: Dept Spec 25A

While these material differences are not great, they should be investigated.

Further, possible differences in weathering of asphalt-sand mortars and their subsequent abrasion by traffic, as well as the manner in which the coarse aggregates polish or disintegrate with age and traffic, are other important factors which need careful study.

The bituminous mixture design for all projects studied falls in the following category:

Coarse aggregate (retained on No. 10 sieve)	50 - 55 %
Fine aggregate (passing No. 10, retained on No. 200 sieve)	30 - 35 %
Filler (passing No. 200 sieve)	5.5 - 6 %
Asphalt cement	5.5 %
Marshall stability	1,500 - 3,000 lb

Photographs of the surface in the traffic lanes of the two gravel projects constructed prior to 1948 show the preponderance of coarse aggregate particles which are in various stages of disintegration. This gradual aggregate disintegration process has evidently caused the continual exposure of new projections with sharp edges which have imparted high skid resistance properties to the surface, irrespective of age and traffic load.

Further evidence of the effect of aggregate performance is supplied by the two surfaces at different ends of one project, which was built since 1948 with 25A specification aggregate under supposedly similar conditions. The coefficient of friction of the two surfaces is decidedly different; in one case it was 0.48, and in the other 0.39. Here again the surface with the better skid resistance characteristics has a higher proportion of coarse aggregate particles in various stages of disintegration, while in the case of the surface with the lower coefficient of friction the aggregate particles appear sound and smooth with less evidence of disintegration and displacement.

The effect of traffic on the skid resistance of pavement surfaces can be understood readily by comparing the results of sliding tests made in adjacent passing and traffic lanes where all factors may be assumed reasonably constant except for the extent of traffic coverage.

Comparative results for eight projects, including one bituminous surface with gravel aggregate, four surfaces containing limestone aggregate, and three portland cement concrete pavement surfaces, show differences in coefficient of friction values between the two lanes amounted to as much as 36 percent.

Typical pavement surfaces in the traffic and passing lanes of several projects show similar surface conditions in adjacent passing and traffic lanes. In the traffic lanes the coarse aggregates, both gravel and limestone, have become worn and polished to varying degrees and the bituminous matrix is only slightly depressed around and between the coarse aggregate particles. The coarser sand particles in the matrix are also worn smooth and flat. In the passing lanes, however, the coarse aggregate particles and the sand particles in the matrix still retain a high degree of angularity and the difference in elevation of the projections of the coarse aggregates and matrix is much more pronounced. The pictorial evidence clearly demonstrates the effect of heavy traffic on bituminous concrete surfaces and why, in many cases, they become slippery when wet.

Portland cement concrete pavement surfaces were not included in the original purpose of the investigation. However, sliding tests were made on several such projects in the course of the work for comparative study. Data indicate clearly that portland cement concrete surfaces, irrespective of mixture composition, can become increasingly slippery with time when subjected to heavy traffic conditions.

Appendix

Figure A shows the force system on the car under the conditions of calibration and test. It was shown in Figure 1, that the stress in the torque tube depends upon the value of skid resistance F_6. Thus, for calibration purposes, the stress in the torque tube is read for a range of values of F_1, measured independently by a traction dynamometer.

However, when the rear wheel is sliding, and the coefficient of friction is being measured, it is evident that the reaction F_5 varies, owing to the effect of the couple

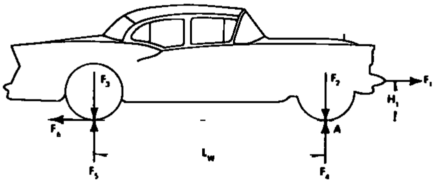

Figure A. Force diagram of car.

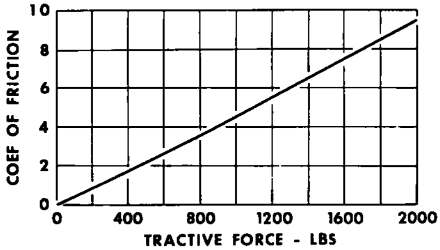

Figure B. Calibration curve for coefficient of friction from tractive force readings.

F_1H_1. The force moment equations are as follows, with the forces as indicated in Figure A and with positive moments in clockwise sense.

$$\Sigma F_X = 0 = -F_1 + F_6, \text{ therefore } F_1 = F_6$$

$$\Sigma M_A = 0 = F_5L_W - F_3L_W + FH_1$$

$$F_5 = F_3 - \frac{H_1}{L_W} F_1 = F_3 - \frac{H_1}{L_W} F_6$$

Coefficient of friction is defined as:

$$\mu = \frac{F_6}{F_5} = \frac{F_6}{F_3 - \frac{H_1}{L_W} F_6}$$

With known constant values of F_3, H_1, and L_W with this vehicle, the coefficient of friction is calculated for values of tractive force, F_6, indicated by stress observations. Figure B shows the coefficient of friction as a function of the tractive force.

HAROLD L. MICHAEL, Closure — The material presented by Skeels, Stonex, and Finney is valuable and, in effect, is a paper itself rather than a discussion.

As pointed out in the discussion and in the paper, one of the factors contributing to pavement slipperiness is the polishing action of the pavement surface. The discussers present concrete evidence of how and why this occurs. As stated, "the most hopeful steps for significant progress lie in the area of developing pavement surfaces which do not become polished under abrasive wear." This certainly is an important factor.

The tests referred to in the discussion develop comparisons similar to tests being performed in Indiana by the Joint Highway Research Project in cooperation with the State Highway Department. These semi-annual tests on a portland cement concrete and a bituminous concrete surface were initiated in 1954 when the surfaces were new. The information to date confirms the observations noted by the discussers, specifically that the surface becomes increasingly slippery with the passage of time and travel and that the traffic lane on dual-lane divided-lane highways is less skid-resistant than the passing lane. These tests are planned for continuance for several years and more information will be developed.

The Indiana studies also have shown that aggregates tend to polish and become slippery in varying amounts: some polish easily, others remain almost unaffected by traffic action. As the discussers indicate, much information is still required before maximum safety can be designed into highways.

Speeds of Passenger Cars on Wet and Dry Pavements

WALTER R. STOHNER, Assistant Civil Engineer
Vehicle Operation Section, Bureau of Highway Planning
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● WET pavements offer less skid resistance than do dry pavements. Do drivers realize this fact and govern their speed accordingly? A study of speeds on wet and dry pavements was conducted in New York, in the spring of 1954, in an attempt to obtain factual information on the subject.

STUDY PROCEDURE

All study locations were on open rural highways removed from the influence of intersections and with a minimum of interference from roadside development. Four sites on 2-lane pavements and one site on a 4-lane divided pavement were selected with horizontal curves of various degrees immediately adjacent to fairly long tangents. Curvature varied from 2½ to 9 degrees. The grades were level or nearly so on the tangent, around the curve and for a considerable distance beyond. This minimized the effect of grades on speed and provided for somewhat similar effects of the curvature on the speeds of vehicles from either direction. Two of the 2-lane sites had blacktop surfaces. The geometric characteristics of the sites studied are included in Table 1.

Observations on wet pavements were taken first. On all occasions, the surfaces were wet and rain was falling from a sprinkle to moderately heavy rain (Table 1). On no occasion did it rain so hard as to affect visibility appreciably. Observations on dry pavements were taken a few weeks later at the same time and weekday corresponding to the wet observations except, at one study location, where additional observations required for the wet pavement condition were taken after the observations for the dry pavement condition.

The number of vehicles recorded varied considerably from site to site as the wet pavement studies were dependent on continued rain. The commercial vehicles recorded were excluded from the study as they were too small in number to permit statistical analysis. Therefore, the analysis includes only passenger cars and, to insure that none was influenced by others, those following another vehicle within a time spacing of nine seconds were excluded. Table 1 shows the number of free-moving and meeting passenger cars in the sample which were used for analysis.

Analysis of Free-Moving Passenger Car Speeds

Passenger car speeds were tabulated according to the site, whether on tangent or curve location, for both wet and dry pavement conditions. Cumulative speed distribution curves were plotted for each location studied from these data (Figures 1-5).

These figures show that there is very little difference in the speed distributions of free-moving passenger cars on wet and dry pavements.

TABLE 1

DESCRIPTION OF STUDY LOCATIONS AND CONDITIONS OF STUDY

								Wet Pavement				Dry Pavement			
			Pavement		Shoulder Width	Curvature	Super-Elevation	Cars	Duration of Study	Precipitation[a]		Cars	Duration of Study	Precipitation[a]	
Site	Route	Location	Width	Type	ft	Deg-Min	ft/ft	Number	hr	Date Mo-Day	Amt in	Number	hr	Date Mo-Day	Amt in
1-2D-1	9	Pottersville	20'	Macadam	4	9 - 0	0 10	144	2 00	5- 3	0 11	132	1 75	6- 7	0
1-2D-2	9	N Hudson	20'	Concrete	6	5 - 30	0 04	131	3 00	5- 3 5- 4	0 11 0 50	129	2 50	6- 8	0
2-2D-2	162	Rural Grove	22'	Concrete	8	4 - 0	0 04	103	4 00	4-14 4-28 6- 2	0 0 2 65	113	3 50	4-14 5- 5	0 0
8-2D-1	9H	Valatia	24'	Blacktop	8	2 - 30	0 01	165	1 75	4-27	0 15	196	3 00	5-18	0
1-2D-7	9	Rice's Corners	48'	Divided Concrete	10	2 - 30	0 02	258	3 75	4-27 4-23	0 20 0 46	360	3 75	5-18 5- 7	0

[a] Records from US Weather Bureau for Station nearest the site Weather Bureau reports may show variations from conditions encountered at the sites because of scattered shower activity prevalent during the spring and summer

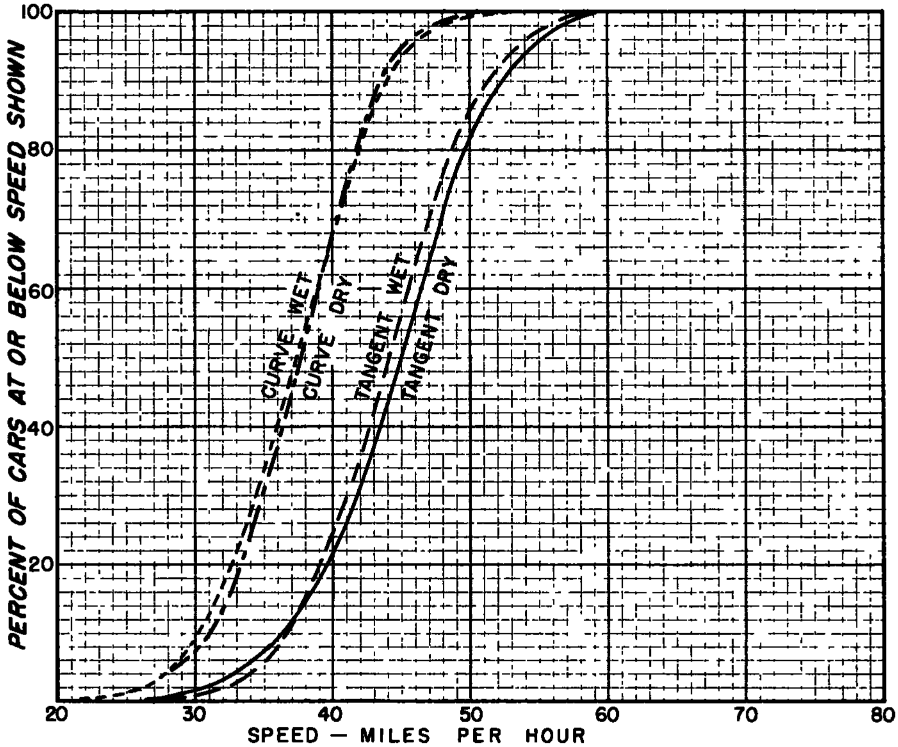

Figure 1. Distribution of speeds at Site 1-2D-1.

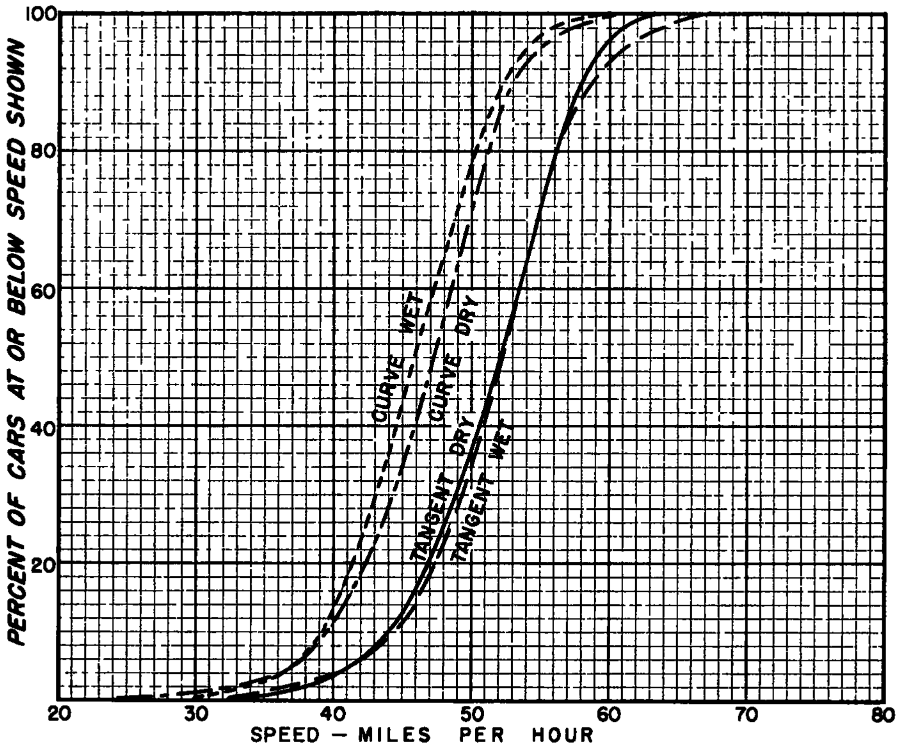

Figure 2. Distribution of speeds at Site 1-2D-2.

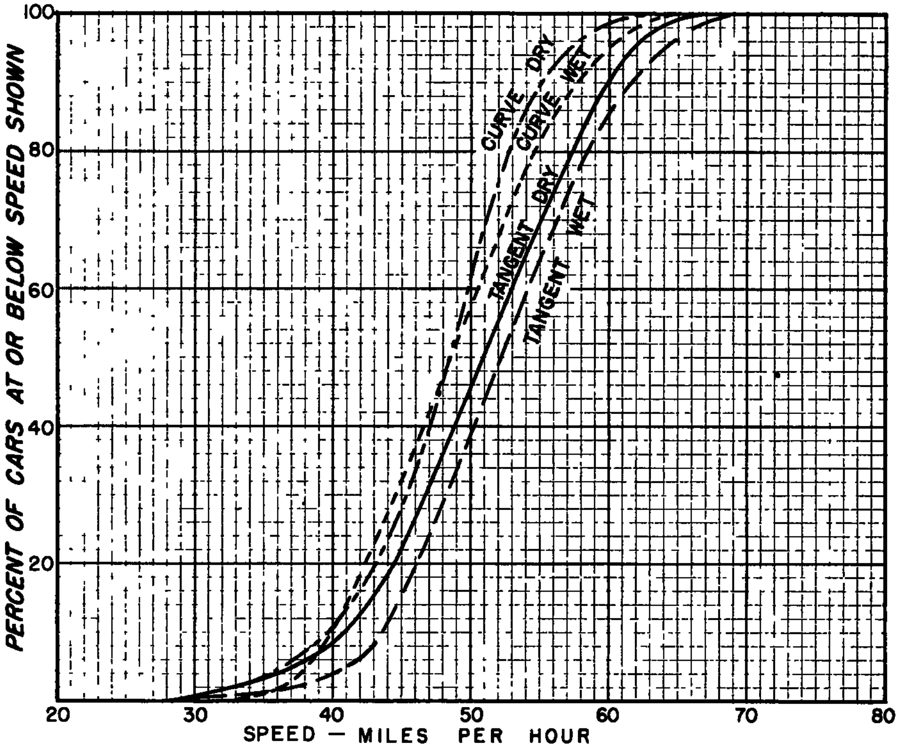

Figure 3. Distribution of speeds at Site 2-2D-2.

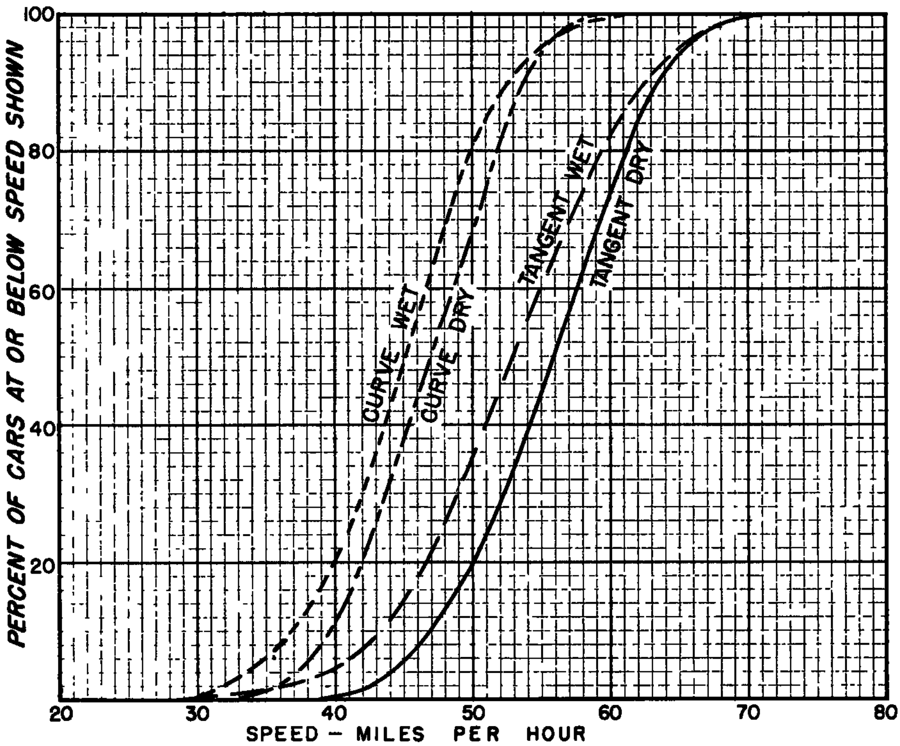

Figure 4. Distribution of speeds at Site 8-2D-1.

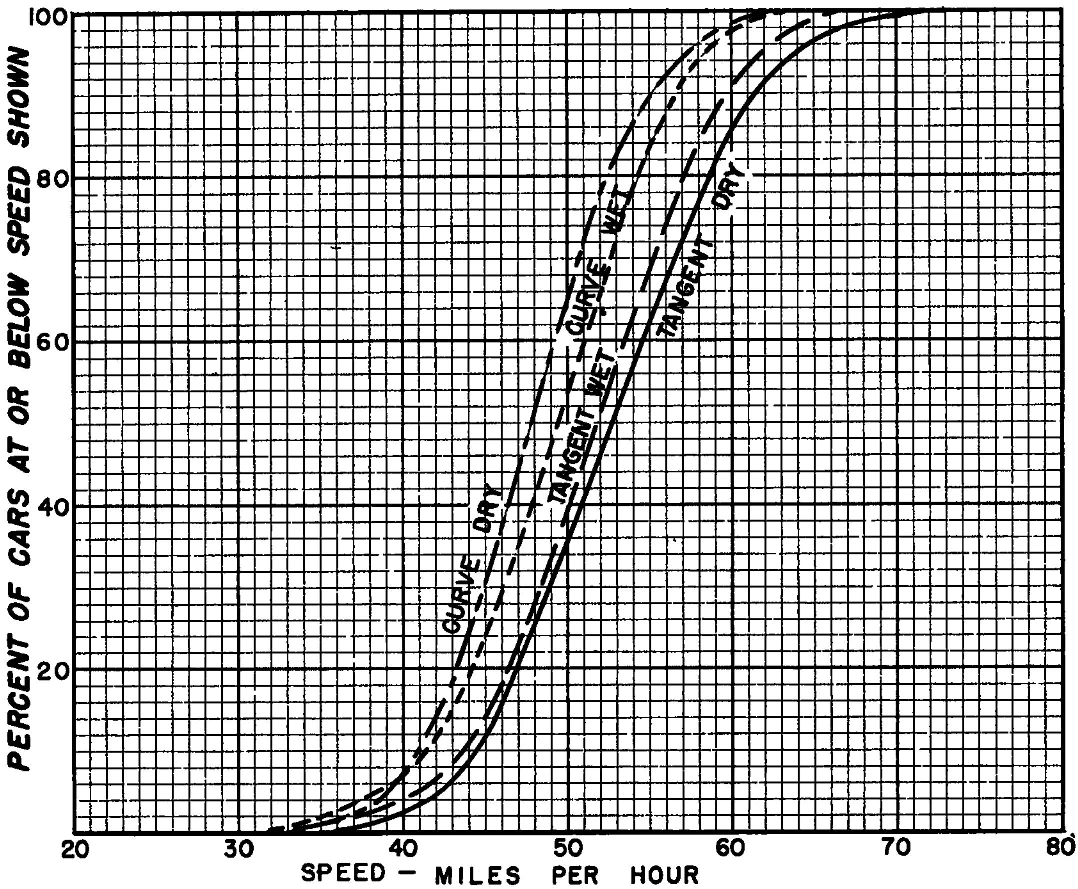

Figure 5. Distribution of speeds at Site 1-2D-7.

Data in Table 2 shows the average speeds, various percentile speeds and maximum speeds observed on the tangent locations at the five sites studied. At these locations, the maximum difference between the average speeds on wet and dry pavements was 2. 8 mph. The average difference was 1. 4 mph. At two of the five tangent locations, the average speeds were higher on wet pavements than on dry pavements. Also, at three of the five tangent locations, the maximum speeds were higher on wet pavements than on dry pavements. It is to be noted that the various percentile speeds show the same general relations as the average speeds.

Data in Table 3 shows the average speeds, various percentile speeds and maximum speeds observed on the curve locations at the five sites studied. At these locations, the maximum difference between the average speeds on wet and dry pavements was 1. 8 mph. The average difference was about 1 mph. At two of the five curve locations, the average speeds were higher on wet pavements than on dry pavements. Also, at three of the five curve locations, the maximum speeds were higher on wet pavements than on dry pavements. Again it is noted that the various percentile speeds show the same general relations as the average speeds.

From an examination of the speed data in Tables 2 and 3 and Figures 1 to 5, it will be found that drivers of free-moving passenger cars operated at about the same

TABLE 2

SPEED DATA RECORDED AT THE TANGENT LOCATIONS

Site	Pavement Type	Surface Condition	Tangent Speed Average mph	70% mph	85% mph	90% mph	95% mph	Maximum mph
1-2D-1	Macadam	Dry	45 27	48 0	50 8	52 5	54 7	59
		Wet	44 64	47 2	49 9	51.4	53 5	61
1-2D-2	Concrete	Dry	52 11	54 8	57 2	58 3	59 8	64
		Wet	52 45	54 9	57 7	59 0	61 5	71
2-2D-2	Concrete	Dry	51 01	55 0	58 4	59 7	61 5	66
		Wet	52 78	56 3	59 7	61 5	64 2	70
8-2D-1	Blacktop	Dry	56 13	59 5	62 4	63 7	65 6	82
		Wet	53 37	57 1	60 9	62 5	65 2	75
1-2D-7	Divided Concrete	Dry	53 36	56 7	59 8	61 5	64 0	77
		Wet	52 08	55 4	58 3	59 7	61 8	66

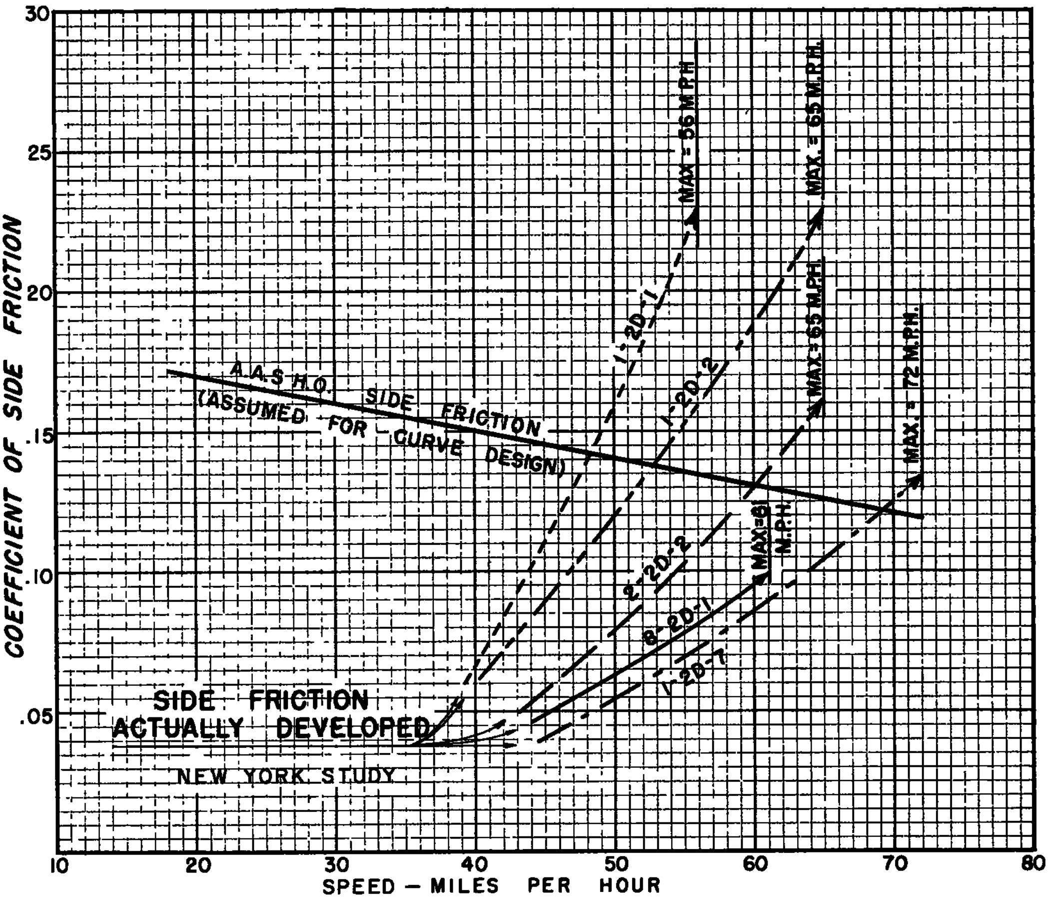

Figure 6. AASHO maximum side friction factors assumed for curve design and coefficients of side friction actually developed at various speeds on curves.

speed on wet pavements as on dry pavements. If the above data, obtained at the five study sites, are typical of operation on the majority of our open rural highways, design speed for wet pavements should be the same as that for dry pavements.

Percent Overdriving AASHO Recommended Maximum Side Friction Factors Assumed for Curve Design

From the data recorded for this study and by using the following basic formula, the coefficients of side friction developed on the horizontal curves included in this study were determined:

$$F = \frac{0.067V^2}{R} - S$$

in which
F = Coefficient of side friction;
V = Speed, mph;
R = Radius of curve, feet; and
S = Superelevation, feet per foot.

These calculated coefficients of side friction for the various speeds on each of the five curve locations studied are shown in Table 3 and have been plotted in Figure 6 with the AASHO recommended maximum side friction factors assumed for curve design.[1]

[1] A Policy on Geometric Design of Rural Highways, American Association of State Highway Officials, 1954, p. 439, Figure III-4.

TABLE 3

SPEED DATA AND COEFFICIENTS OF SIDE FRICTION DEVELOPED ON CURVES

Site	Pavement Type	Surface Condition	Curvature deg-min	Curve Speed Average mph	Curve Speed 70% mph	Curve Speed 85% mph	Curve Speed 90% mph	Curve Speed 95% mph	Curve Speed Maximum mph	Coefficient of side friction developed at indicated speed Average mph	70% mph	85% mph	90% mph	95% mph	Maximum mph	Percent of all cars overdriving design coefficient of side friction[a]
1-2D-1	Macadam	Dry	9 - 0	38 18	40 5	42 7	43 7	45 3	53	0 05	0 07	0 09	0 10	0 11	0 20	1 5
		Wet		37 94	40 5	43 0	44 1	45 7	56	0 05	0 07	0 09	0 10	0 12	0 23	2 1
1-2D-2	Concrete	Dry	5 - 30	47 43	50 1	52 2	53 3	55 3	65	0 10	0 12	0 13	0 14	0 16	0 23	14 0
		Wet		46 45	48 8	51 5	52 6	54 5	64	0 10	0 11	0 13	0 14	0 15	0 22	12 2
2-2D-2	Concrete	Dry	4 - 0	48 44	51 2	53 6	55 0	57 2	63	0 07	0 09	0 10	0 11	0 12	0 15	3 5
		Wet		49 03	52 5	55 8	57 5	60 0	65	0 08	0 09	0 11	0 12	0 13	0 16	8 7
8-2D-1	Blacktop	Dry	2 - 30	47 29	50 4	52 8	53 8	55 2	59	0 06	0 06	0 07	0 07	0 08	0 09	0
		Wet		45 45	48 3	51 2	52 8	55 1	61	0 05	0 06	0 07	0 07	0 08	0 10	0
1-2D-7	Divided Concrete	Dry	2 - 30	48 50	50 8	53 8	55 3	57 4	72	0 05	0 06	0 07	0 07	0 08	0 13	0 3
		Wet		49 77	52 7	55 5	56 8	58 5	67	0 05	0 06	0 07	0 08	0 08	0 11	0

[a] AASHO recommended maximum side friction factors assumed for curve design

Referring to Figure 6, the point of intersection of each curve for the plottings of the coefficients of side friction actually developed, with the straight line representing the AASHO maximum side friction factors assumed for curve design, indicate the speeds beyond which cars were overdriving the AASHO recommended design coefficient of side friction for each curve location. To determine the percentage of the cars overdriving the AASHO recommended design coefficient of side friction, these speeds were applied to the appropriate cumulative distribution of speed curve, shown in Figures 1 to 5. These values are shown in the last column of Table 3.

The percentage of passenger cars shown overdriving the AASHO recommended design coefficient of side friction appears to be higher for the concrete pavements than the blacktop surfaces. Drivers apparently realize that the side friction on blacktop surfaces, whether they be wet or dry, is lower than it is on concrete pavements. However, the lack of any significant variation in speed indicates that they recognize no difference in friction factor between wet and dry pavements whether the pavement be blacktop or concrete.

CONCLUSIONS

1. The speeds of free-moving passenger cars on wet pavements are not appreciably lower than those on dry pavements. This was found to be true on macadam and blacktop surfaces as well as on the portland cement concrete surfaces.

2. Even though the speed of free-moving passenger cars is not generally reduced on wet pavements, an analysis of speed and the coefficients of side friction actually developed shows that most drivers of passenger cars do not exceed the design coefficient of side friction recommended by the AASHO.

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