HIGHWAY RESEARCH RECORD

Number 376

Anti-Skid Program Management and Related Papers

22 Reports

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FOREWORD

Part 1 of this RECORD constitutes the proceedings of the Anti-Skid Program Workshop held January 17 and 18, 1971. Some of the papers were prepared for and served as a basis for subsequent discussion at the workshop. Participants were invited to prepare written briefs of their remarks, and many did so. The discussion by Ralph Moyer provides background and summary of past work in the field and is used here as an introduction to the workshop proceedings. Burton Marsh prepared a summary of the workshop including a critique and suggested future work at the concluding plenary session. It is included near the beginning of the volume to enable the reader to gain a quick overview of the workshop sessions.

Anti-skid programs by state highway departments, the Federal Highway Administration, and the National Highway Traffic Safety Administration are being closely monitored by appropriate congressional committees. There is every indication that expenditures for research and implementation of research findings will expand at least fourfold in the next few years. The workshop was organized to assist researchers and administrators to take advantage of current knowledge in organizing anti-skid programs.

The papers in Part 2 were reviewed as part of the offerings for the 50th Annual Meeting and were recommended for publication in this RECORD as material complementary to the proceedings of the workshop. Several aspects of the vehicle-pavement interaction have been examined.

The paper by Brach presents a general equation of motion of an automobile skidding on a grade. The equation includes effects of aerodynamic drag and friction coefficients that vary with speed. Solutions are demonstrated that yield friction as a function of speed and that are useful in accident investigation where estimates of initial speed of skidding vehicle are desired.

A comparison of pavement friction measurements taken in the cornering-slip and skid modes is presented in the paper by Gallaway and Rose. Favorable correlation of results was obtained over a wide range of speeds and surface friction levels when similar tire-tread configurations were used. The authors emphasize the importance of providing adequate drainage in the tire-pavement contact area.

The final two papers deal with skid-resistance properties of aggregates. Mullen, Dahir, and Barnes report on the development of a circular track method and a jar mill method for pre-evaluating skid-resistance properties of aggregates in the laboratory. Laboratory skid testing of pavement samples evaluated by the first method and of pavement samples prepared from aggregates evaluated by the second method indicated a linear correlation as well as the same order of rating of aggregates. In the final study, aggregates rated by these two methods were examined by Dahir and Mullen to determine factors that influence observed differences in laboratory determined skid-resistance properties. The authors found no general correlation between physical properties of aggregates and skid resistance but did observe correlations within some petrographic groups. Other conclusions are reported.

Highway engineers, administrators, materials suppliers, and legislators will find material in this RECORD of interest.

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PART 1

ANTI-SKID PROGRAM MANAGEMENT

HISTORY OF ANTI-SKID PROGRAM MANAGEMENT IN THE UNITED STATES

Ralph A. Moyer, University of California, Berkeley

•MANY of the topics discussed at the Anti-Skid Program Workshop were thoroughly investigated in a comprehensive research program on skidding characteristics of automobile tires that I initiated in 1932 at Iowa State University. The measurement of the skid resistance of pavements of all types—dry and wet, ice- and snow-covered—using a two-wheeled skid test trailer operating at a wide range of speeds was a major pioneering breakthrough in this comprehensive research program. Another feature of this program was theoretical analyses, confirmed by road tests, to determine the friction requirements and the friction attainable at the tire-road interface in braking, cornering, acceleration, and gradability of cars and trucks. The major objective of the theoretical analyses and the road tests was to develop factually based geometric highway design standards to replace the outmoded rule-of-thumb standards used by state highway departments at that time (1, 2, 3).

A two-wheeled skid test trailer was developed in this early research to measure the coefficients of friction on dry and wet pavements in locked-wheel braking and impending skid braking and to measure the side skid coefficients of friction. The soundness of this procedure is indicated by the fact that in the United States today a two-wheeled skid test trailer has been adopted as a standard test method by the ASTM Committee E-17 to measure the skid resistance of pavements in terms of locked-wheel-braking coefficients of friction. In Great Britain, the Road Research Laboratory has developed a routine testing machine consisting of a single-unit truck carrying a fifth wheel to measure the sideway-force coefficient of friction at speeds up to 60 mph.

The British Road Research Laboratory currently is recommending minimum sideway-force coefficients of friction of 0.5 to 30 mph and 0.45 at 50 mph for average sites on motorways or on other high-speed roads. Kummer and Meyer (4) recommended a minimum interim skid-resistance standard using the ASTM skid test trailer locked-wheel-braking test method that provided for coefficients of friction of 0.37 measured at a speed of 40 mph for a highway with a mean traffic speed of 50 mph and of 0.41 measured at a speed of 40 mph for a highway with a mean traffic speed of 60 mph. These decisions substantiate recommendations made by me almost 40 years ago (3).

The development of comprehensive anti-skid programs, however, is not solely a matter of developing test method and skid-resistance standards. The research and management problems in developing comprehensive anti-skid programs are very complex. Progress in this area has been insufficient. The anti-skid programs being developed by the National Highway Safety Bureau, the U. S. Department of Transportation, and the state highway departments in each of the 50 states should now finally bring about improved traffic safety. Two important features of comprehensive statewide anti-skid programs should include (a) annual accident studies covering all streets and highways to identify high-accident locations and sections of streets and highways with high accident rates where slippery-when-wet pavements may be a major contributing factor and (b) an inventory of the skid resistance of all streets and highways and, especially, the measurement of the skid resistance of the pavements that have been identified as high-accident locations or that have a high accident rate under wet or icy pavement conditions.

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It is difficult to identify skidding accidents by the use of the computer methods of analysis of traffic accidents because no provision is made on the standard traffic accident report form to classify a given accident specifically as a skidding accident. The weather conditions are identified as clear, raining, or snowing; the road character is shown as straight, sharp curve, turn, or grade; and the road surface is listed as dry, wet, slippery, snowy, or icy. The vehicle condition is listed in terms such as defective brakes or steering, tire puncture or blowout, and worn or smooth tires. The speed of the vehicle or vehicles involved and the accident location are also shown on the accident report form. Many states now have compiled tapes and punch-card records on which all these data are summarized. An analysis of the accident records can then be made to classify and summarize accidents for any given condition or type in a very short time and at a low cost.

Many studies have shown that there is a high degree of correlation between the total number of accidents on a given section of highway and the coefficient of friction as measured by a standard test method. Thus, on dry pavements with coefficients of friction of 0.60 or higher, the normal total accident rate will average 1.00 to 3.00 accidents per million vehicle-miles. On wet pavements with friction coefficients ranging from 0.30 to 0.40 at 50 mph, as measured by the ASTM standard test method, the total accident rate may be doubled to average 2.00 to 6.00 accidents per million vehicle-miles. When the friction coefficients are definitely in the slippery-when-wet range of 0.15 to 0.25, very high accident rates up to 15 and 20 accidents per million vehicle-miles have been reported. I know of no published accident reports listing total accident rates under icy pavement conditions where the coefficients of friction for all unstudded tires at speeds of 10 to 30 mph are less than 0.10, but I am confident that the accident rates far exceed those reported for slippery wet pavements.

One of the most important anti-skid studies urgently needed today is a thorough, impartial fact-finding study to reach a satisfactory solution to the studded-tire controversy. Such a study should clearly determine on a factual basis the benefits derived from the use of studded tires in saving lives and in the reduction of traffic accidents under snowy and icy pavement conditions. It should establish the extent to which studded tires increase the accident hazards resulting from the ejection of studs when cars are traveling at high speeds on wet or dry pavements. It should also determine whether the trough type of pavement wear in the wheel tracks caused by studded tires creates (a) a traffic hazard because of its effect on steering control of vehicles or (b) a hydroplaning skidding hazard during heavy rainstorms when the troughs in the wheel tracks on flat grades are filled with water to a depth of ½-in. or more or (c) an added skidding hazard on glare ice when the water in the troughs freezes. The study should determine the feasibility and cost of repairing pavement damage caused by studded tires. Cost-benefit ratios should be established if that is possible.

The investigation should determine the type and extent of use of studded tires in the snow-belt states. Recent reports indicate that the majority of car owners currently equip their cars with studded tires on the rear wheels only to provide increased traction as a replacement for tire chains. The use of studded tires on rear wheels only raises some grave questions in regard to improved performance and added safety provided by the use of studded tires versus the use of snow tires and/or the use of tire chains under icy pavement conditions. Tests conducted by the National Safety Council Winter Driving Hazards Committee have shown consistently that the friction coefficients for unstudded tires on glare ice are in the range of 0.06 to 0.08 at ice temperatures of 15 to 32 F. If studded tires are used on two wheels only, the coefficients of friction are increased by 50 percent; if studded tires are used on all four wheels, the coefficients are doubled. These test results apply to studded tires when new. Test results for worn studded tires have rarely been reported and are not conclusive. Obviously, the coefficients of friction for worn studded tires on icy pavements are lower than those for new studded tires.

It is significant to note that, in the National Safety Council tests of studded tires, the coefficients of friction on glare ice rarely exceed 0.12 at ice temperatures ranging from 15 to 32 F. Many studies have shown that, in normal day-to-day driving on dry pavements, 25 to 50 percent of all vehicles require friction coefficients in the range of 0.30

to 0.40 in braking, cornering, and acceleration. This is why a braking coefficient of 0.40 at 40 mph and a sideway-force coefficient of 0.50 at 30 mph are today recommended as the minimum coefficients of friction for wet pavements by the leading authorities in the United States and Great Britain. This is also why studded tires, which increase the coefficients of friction from 0.08 to 0.12, are not effective in preventing wet-pavement accidents.

It should also be realized that to achieve the maximum potential of studded tires in tire performance on glare ice will require more studs per tire than are currently used in the United States and Canada and will require studded tires to be used on all four wheels according to the National Safety Council tests and tests of studded tires by European testing agencies. Under these conditions it is reasonable to expect that the pavement wear per car will double the current wear rate for cars with studded tires on the rear wheels only.

Published results of friction tests comparing studded tires with snow tires under various snow conditions, such as hard packed, loosely packed, and virgin snow, are very limited. In general, the friction coefficients for all tires range from 0.20 to 0.50. This wide range is due primarily to variations in the type, texture, and crystalline properties of the snow. It has generally been observed that, at snow temperatures in the range of 20 to 30 F, the effect of compaction of the snow by traffic is to form a thin glazing of ice on the packed snow surface for which friction coefficients as low as 0.15 for snow-tread tires have been measured. It is only when the packed snow is coated with a thin glazing of ice that studded tires provide slightly improved performance in braking, cornering, and acceleration when compared with the performance of snow-tread tires. Under loosely packed and virgin snow conditions, studded tires do not provide improved performance when compared with the performance of snow tires.

The California Division of Highways and the California Highway Patrol adopted a policy in 1971 that permits the use of snow-tread tires during snowy pavement conditions but requires tire chains to be carried under those conditions for emergency use and for use when the "chains required" signs are posted. In California and in many other states, an elaborate snow-removal and bare-pavement maintenance program has been developed that has been very effective in reducing winter driving hazards. Although drivers protest the required use of tire chains because of the inconvenience and difficulty frequently experienced installing tire chains, the National Safety Council tests under snow and ice conditions clearly showed a marked superiority in traction and in stopping ability for cars equipped with tire chains compared with cars equipped with studded tires. Only when studded tires were used on all four wheels was improved performance obtained in cornering tests with friction coefficients of 0.10 to 0.12 on glare ice; in similar tests, coefficients of 0.07 to 0.08 on glare ice were obtained for cars equipped with snow tires or with tire chains on rear wheels only.

It is a well-known fact that tire chains are used for the most part only under snowy and icy pavement conditions. Drivers remove the chains as soon as the pavements are free of snow and ice. Thus, pavement wear caused by tire chains has been minimal.

It will be difficult to obtain a satisfactory solution to the studded-tire controversy. Nevertheless, it is important that appropriate government agencies take steps now to make an impartial and thorough investigation of the accidents occurring with studded tires, snow tires, and tire chains under snowy and icy pavement conditions to establish the relative merits of each type of operation. All other factors that have an important bearing on reaching a final decision in the use of studded tires should also be made a part of such an investigation.

In the interim, highway departments should continually improve winter highway bare-pavement maintenance programs by prompt removal of snow and ice, thereby providing the maximum anti-skid safety possible under winter driving conditions. The use of snow-tread tires and of tire chains should be required depending on the snow and ice conditions that require their use. Strict enforcement of speed regulations, e.g., limiting speed to 25 mph under snowy and icy pavement conditions when the "chains required" signs are posted, is an important feature of a winter driving anti-skid program. The recommendations proposed by the participants at the Anti-Skid Workshop and the HRB

Annual Meeting should contribute to the development of an outstanding anti-skid management program, not only for safe winter driving but also for safe year-round driving.

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SUMMARY OF WORKSHOP ON ANTI-SKID PROGRAM MANAGEMENT

Burton W. Marsh, Consulting Engineer, Washington, D. C.

•PARTICIPANTS in the Workshop on Anti-Skid Program Management stressed that skidding and its many related factors constitute a very serious and complex challenge in highway transportation. Skidding is clearly a major problem that is increasing and that warrants commitment of very substantial manpower, funds, and top-level support.

Whitehurst stated that the basic purpose for a comprehensive anti-skid program is to provide highway users with levels of pavement-tire skid resistance sufficient to permit all appropriate driving maneuvers with reasonable margins of safety. The fact that, under the 1966 Federal Highway Safety Act, the federal government can require a program meeting certain standards should help to attain better progress.

Baldwin's paper on assembly and usage of accident data should be required reading by all who are, or should be, concerned with anti-skid programs. He pointed out that accident records can provide sound guidance to a state in its anti-skid program, although the overall performance level is not yet up to its full potential. For example, only 19 states (of the 37 responding) in 1967 said that they used accident information for the detection of slippery pavement sections. Baldwin showed the correlation between side-force coefficient and skidding accident rate and the relation between rainy weather and accidents.

Ricker described the progressive program of the Pennsylvania highway authorities in using accident data to identify wet-pavement accident locations. He stressed the importance of the kind, quality, and accuracy of information provided in accident reports. In Pennsylvania, determination of accident location is given much attention. Cluster listings and tabulations of sections of highways involving high proportions of accidents in wet weather, as well as "search by hazard," provide information leading to skid-test lists. Any section (at present) with a skid number of less than 30 is treated immediately. Those with a skid number of 30 to 40 are placed on future programs for corrective measures. The Pennsylvania program has not been in effect long enough to make significant "after" studies.

Goodwin stressed that a sound management approach can be a major contributor to success in highway safety programs. Inspection of highway characteristics and inventory of pavements as to level of skid resistance must be continuing programs in each state. The methods and importance of vehicular inspection are important. Inspection programs should be useful in achieving the objectives of an anti-skid program and should be reviewed to ascertain their effectiveness.

Oliver presented legal concepts and case law relating to wet-weather conditions. His paper should receive the careful attention of state, county, and city attorneys as well as other officials dealing with anti-skid matters. Because government immunity has been increasingly challenged, there has been a substantial increase in litigation against government agencies in performance of proprietary functions. Because of the difficult financial problems of state and local governments, Oliver believes that courts will likely give more attention or emphasis to contributory negligence by the driver in relation to driver recovery of damages. Decisions allowing recovery for accidents on slippery roads have been based on not just the presence of slick conditions but the presence of a defect or obstruction. Oliver pointed out a number of other factors of importance in such litigation. The matter of legal responsibility of highway transportation agencies and officials is very important. One of the workshop groups recommended

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that the Highway Research Board consider sponsoring a meeting on that subject. A digest concept was proposed.

Ivey, Keese, McNeill, and Brenner stated that there currently exist mismatches among driver capability, vehicle capability, and highway conditions. Elimination of such mismatches should be a primary goal of any program to reduce skid-initiated accidents. The interaction of driver, vehicle, and roadway has not been successfully synthesized, although it is critically needed. In research, relatively minor emphasis has been placed on interaction of variables. Almost all investigations have been based on empirical information, and there have been relatively few attempts to quantify effects theoretically. Nonetheless some important developments are occurring, and there is now much technology ready to be applied.

Mahone and Shaffer cited four major ingredients of a successful corrective program: (a) responsible attitude toward providing skid-resistant roads; (b) knowledge of friction needed; (c) reliable friction measuring method; and (d) technical knowledge of materials and methods, such as superelevation, spot improvements, resurfacing, grooving, elimination of sharp curves, and other techniques, used in providing skid-resistant roads. They mentioned British goals of skid numbers of 55 for high traffic roads, 50 for medium traffic, and 45 for lesser traffic. They also indicated that in Virginia lower skid numbers than expected are being obtained on heavy traffic roads.

The following points were raised repeatedly during the workshop:

1. There is a basic need for a much clearer understanding and quantification of anti-skid needs of drivers in carrying out the various driving maneuvers such as deceleration, acceleration, cornering, cornering while decelerating, overtaking, and passing;
2. This basic need can only be met when satisfactory and reliable measurements are made;
3. Tentative minimum levels of skid resistance for various situations and conditions must be set; and
4. Those responsible for highway safety must decide, on a local basis, how best to achieve and maintain at least minimum standards in a continuing anti-skid program.

In my opinion, two areas that did not receive enough attention at the workshop are the precautionary actions that drivers can take to decrease skidding accidents and the kinds of materials used in tires and their effects on skidding.

Because there are so many factors that influence an anti-skid program, priorities should be established for actions that experts in the anti-skid field need to take. The following are but a few of the many that should be considered.

1. Establish and maintain support for the program by improving communications among administrative, legislative, and judicial officials, researchers, interested groups, and the general public;
2. Develop techniques that will decrease the threat of skidding accidents;
3. Make research findings available as soon as possible, and implement research results that already exist and are applicable;
4. Prepare "white paper" explaining the importance of anti-skid work and justifying the need for more research; and
5. Summarize recommendations of this workshop so that they can be used as guidelines by highway transportation administrators and maintenance men during 1972.

PROBLEMS ASSOCIATED WITH ANTI-SKID PROGRAM MANAGEMENT

E. A. Whitehurst, Transportation Research Center, Ohio State University

•ALTHOUGH the federal government requires states to maintain an anti-skid program, the success of this program depends on the actions of the state and local governments charged with administering the program. Successful management of such a program ultimately depends on the dedication of those people who are responsible for ensuring highway safety.

The 1969 motor vehicle accident statistics released by the National Safety Council (1) may encourage such dedication. In 1969 motor vehicle accidents accounted for 56,400 deaths, 2,000,000 injuries that disabled the victims beyond the day of the accident, and costs totaling $12.2 billion. Figures for the decade are 475,000 deaths, 17,200,000 injuries, and $89.6 billion. It cannot be reasonably claimed that lack of adequate pavement skid resistance contributed significantly to all of the 15.5 million accidents reported in 1969. There is, however, some evidence that a disproportionate percentage of accidents, as related to exposure, occurred on wet pavement surfaces and that an even more disproportionate percentage occurred on snow- or ice-covered surfaces.

Large public works programs cannot be successfully undertaken in today's society without the support of the general public. Therefore, hard-core highway professionals and concerned public officials have a dual job. They must solicit and win the support of the public and then solve the problems discussed at this workshop.

Citizens must be informed of the need for the program and of the benefits they will receive from its successful completion. The average driver must be convinced that his investment of tax dollars in the program will pay dividends in the form of a reduction in the hazards of driving on wet pavements.

Even after adequate motivation both of the public and of those officials responsible for maintaining an anti-skid program has been developed, several questions must first be answered before the program can be undertaken. Two significant ones, not totally unrelated, are, What level of skid resistance is adequate to permit vehicles traveling on the pavement to perform the required maneuvers safely? and How shall the skid resistance of the surface be measured?

FRICTIONAL REQUIREMENTS

The adequacy of the skid resistance of any pavement surface can be measured by the maneuverability of a vehicle passing over that surface. Although this measurement can be calculated accurately on a theoretical basis, the actual value required in any finite instance will be influenced not only by the quality and condition of the surface but also by the quality and condition of the vehicle tires and, often to a large degree, by the behavior of the driver. One widely cited report (2) resulting from a study that gave at least limited consideration to these various requirements recommends that a skid number (defined as the friction coefficient of a tire sliding on a wet pavement times 100) of 37, as measured with a skid trailer in accordance with ASTM Designation E 274 at 40 mph, be considered as the minimum requirement for pavement friction on main rural highways. Although this recommendation has not yet been formally adopted on a national basis by any organization, it appears to reflect the thinking of a number of people or groups who are currently concerned with the measurement of skid resistance and

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who report informally that they consider skid numbers ranging from 35 to 40 as being adequate. The same report, however, suggests that on high-speed roadways where the mean traffic speed is likely to be on the order of 70 mph the skid number measured at 40 mph should be 46. This recommendation is based on the reduction that occurs in skid resistance between tires and a wet pavement surface as the vehicle speed increases. This may not be altogether in accord with the thinking of others (3) who have suggested that a lower skid resistance may be required on a high-speed roadway, where grade and alignment variations are limited and access is controlled, than on a roadway on which vehicle speeds are lower but where steeper grades, sharper curves, and lack of access control are likely to place greater demands on vehicle performance. Studies of actual driver behavior under differing circumstances are extremely limited, and it appears that much more work will be required in this area before a satisfactory rationale for skid resistance criteria can be established.

MEASUREMENT PROBLEMS

The problems associated with the measurement of skid resistance have received far greater attention than have those associated with establishing the desirable level of skid resistance. It is well known that skid resistance has been measured by one technique or another since before the turn of the century (4) and that concentrated studies of the problem and development of equipment have been in progress since the early 1960s (5, 6, 7). Although many techniques have been investigated, the practice of measuring pavement skid resistance using a type of locked-wheel sliding test has been overwhelmingly adopted in the United States. It is appreciated that this is the mode of operation least often experienced by a vehicle and, indeed, is the mode that should be avoided. The emergence of such methods as the predominant ones, however, has apparently been based on ease of performance, reduced required sophistication of test equipment, and a feeling that the locked-wheel mode, although not the normal or desired mode of operation, is in fact the mode existing from the time the vehicle loses traction (and becomes uncontrollable) to the time it regains it or comes to a stop.

The most widely used standardized method of test is that specified in ASTM Designation E 247 in which one or both wheels of a towed trailer may be locked and appropriate measurements can be made from the towing vehicle. Various organizations have built a considerable number of trailers that appear to be amenable to this test method. Two sizable skid correlation studies (8, 9) have been held in which a number of such trailers were involved in tests on surfaces having a wide range of skid resistance. These studies have generally achieved reasonable agreement of test results where the trailers are well-designed and well-built and operated by trained crews and when the test pavement surfaces are watered from an external source so that all trailers are operated under the same condition of pavement wetness. When external watering is discontinued, however, each trailer provides its own wetting of the test surface, and results are extremely variable. Some trailers occasionally measure skid numbers twice as great as those measured by other trailers on the same surface and within a time interval of a few minutes. There is some evidence that, even when the same quantity of water per unit distance traveled is supplied, variation in nozzle design, nozzle location, and even aerodynamic effects of the towing vehicle may have a significant effect on the thickness of the water film actually deposited in front of the test tire and, therefore, on the test results. In establishing an anti-skid program, or programs, it will be imperative that standards be established for the application of water to the pavement surface during testing and that methods be developed for evaluating compliance with such standards. It appears that a great deal of work remains to be done in this area.

Another decision that must be made in establishing an anti-skid program is where, when, and how often measurements shall be made. For a full program, this is likely to involve three separate phases of the problem.

One phase is the conduct of skid resistance measurements at locations where an abnormally high number of accidents occur or at sites that are reported by the public, or particularly by police officers, as being slippery. These measurements make it possible to determine whether inadequate skid resistance is a contributory factor to the accident rate. This is an entirely rational approach and should result in the identification of surfaces that are clearly inadequate and need remedial treatment. It does not, however, provide information on which long-range planning for major rehabilitation may be based.

An inventory of pavement skid resistance will be required to provide the necessary basis for such planning. This implies that every pavement surface within the system covered by the program should be tested periodically to determine the rate of development of slipperiness and to estimate when remedial treatment would be required. Such a program requires definition of the words "every surface" and "periodically" because both the adequacy of the data collected and the magnitude of the effort required will depend on these definitions. At one extreme, it has been suggested that every highway should be tested at intervals of 1 mile or less and that the tests should be repeated annually. At the other extreme, it has been suggested that one series of tests should be made within the limits of each construction project within the highway system involved and that the entire system within a state should be tested over a 3-year period. Again, there has not been broad agreement as to what will constitute an adequate survey, and there has not been sufficient effort to achieve one. Such decisions must, however, be made before an adequate anti-skid program can be started.

A final phase of a full anti-skid program is acceptance testing of new construction or of remedial treatments applied to pavements determined to be slippery. In the past some pavements have been constructed or resurfaced that were inadequate in skid resistance upon completion. If an anti-skid program is to be successful, such occurrences must be avoided in the future.

REMEDIAL MEASURES

After the decisions have been made as to what constitutes an adequate level of skid resistance and how, when, and where it shall be measured, there remains the problem of achieving and retaining the selected level. The present state of knowledge in this field suggests that the highway engineer, whether he is concerned with initial construction or restorative treatments, can approach the problem in two ways: choice of aggregate and choice of pavement surface texture.

Numerous studies have shown that, other parameters being the same, the type of aggregate exposed in the pavement surface plays an important role—sometimes a predominant role—in the achieved skid resistance (7, 10). These studies have been sufficiently extensive so that in many parts of the country the relative contribution of all readily available aggregates to skid resistance may be known. Where this is not the case, laboratory devices have been developed (11, 12) that permit rapid evaluation of the relative skid resistance of potential aggregates when used in paving mixes. Laboratory evaluation can facilitate decision-making and does not involve expensive and time-consuming field experimentation.

Some areas have an abundance of aggregate that is relatively polish resistant and suitable for construction of pavement surfaces having high skid resistance. In other areas, supplies of such aggregates are limited, are therefore generally more expensive, and must be used selectively. Where such aggregates are nonexistent, attention should be given to the possible use of materials that have not in the past been widely used as general paving aggregates. Some of these materials, particularly some of the expanded aggregates (13), have shown considerable promise of producing satisfactory skid-resistant pavements. Some highway departments that have experimented with materials of this nature have experienced difficulties in arriving at equitable units for payment (due to the low specific gravity of the aggregates) and in specifying proprietary materials. However, solutions to problems such as those may be more readily achieved than solution to the larger problem of providing adequate pavement skid resistance.

Texture of the pavement surface also plays a significant role in pavement slipperiness although it has had less extensive investigation than aggregate (14). Probably the ultimate solution to wet-pavement friction is to allow as little water between the tire and the pavement as is possible. This can readily be demonstrated by comparing skidding performances of vehicles equipped with smooth tires to those equipped with tires having a deep tread (stopping distances at 40 mph may be more than 100 ft longer for the car equipped with smooth tires). The primary reason for the use of deep-tread designs in tire manufacture is that the grooves in the tread provide an escape route for water. It is generally appreciated that a similar function can be performed by the texturing of the pavement surface, i.e., the provision of escape routes to facilitate removal of water at the tire-pavement interface. The highway engineer may influence the texture of either a new pavement construction or a remedial surface treatment by his choice of aggregate gradation, mix design, and finishing treatment in order to affect beneficially the skid resistance of the resulting surface.

The engineer may also give consideration to the possibility of achieving the desired improvement through altering the existing surface texture rather than by replacing the surface. Of the techniques, such as etching, chipping, and grooving, that have been attempted to date, only grooving has been shown to produce reasonably lasting effects. In a number of cases, the grooving of hazardous sections has been followed by a significant reduction in accident occurrence (15). The reason for this improved performance may not be fully understood because in some cases the grooving operation has not resulted in an equivalent increase in measured skid resistance.

The engineer should always be aware that actions taken to improve the skid resistance of a surface over a short distance, as in spot improvement programs or other forms of maintenance, may create worse hazards than those they were intended to correct. For example, a remedial treatment applied to a curve should be continued at least sufficiently far past the point of tangency of the curve to ensure that a vehicle rounding the curve on something other than the true uniform radius of its lane will have completed its cornering maneuver before it leaves the treated area. If not, the driver may experience a situation where he has a suddenly decreased tractive capacity just as he is making his greatest demand on his vehicle for tractive performance. Similarly, when long patches are applied to a pavement surface, they should be extended over the full width of that surface. It is quite common in many areas to find patches several hundred feet in length that extend over only one-third to one-half of the pavement width, or similar patches covering only one lane of two adjacent traffic lanes. Under such circumstances the driver may be expected, and indeed may be required, to drive with two wheels of the vehicle on the patch and two on the previously existing surface. If under these conditions the surface is wet and the driver either brakes or accelerates heavily, loss of control is almost certain to occur. In any full anti-skid program, policies should be developed and enforced to eliminate the construction of such hazardous situations.

The highway engineer must also consider the effect of geometric design on traction requirements. The provision of adequate pavement surface drainage will minimize the accumulation of water and, hence, improve the skid resistance performance of any surface. In new construction, provision for minimum grades and maximum radius of curvature will decrease the tractive requirements of the motorist. Although it is more expensive than remedial surface treatment, improvement of pavement geometry should be considered at sites where accidents occur frequently.

PERIPHERAL CONCERNS

To this point, the discussion has dealt with aspects of an anti-skid program that may reasonably be considered to be within the control of the highway engineer or administrator. There are a number of peripheral areas over which he does not have control but in which he should have a keen interest and should exercise some influence in their mangement. The most obvious of these has to do with tires.

A single body does not really have a coefficient of friction. It is only when two bodies move in contact with each other that friction is developed. Thus, when we speak of the coefficient of friction (or the skid number) of a pavement surface, we are in fact speaking of the frictional resistance between that surface and something that is moving against it—normally a tire. A change in the properties or condition of either of the bodies can, and usually will, significantly change the coefficient of friction between them.

The automobile tire might be considered to be one of man's great compromises. What does a vehicle owner want when he purchases a tire? It is probably safe to say that as a minimum he would like the tires he purchases to be (a) so durable that they will last for the lifetime of his vehicle, (b) so skid resistant that they will stop the vehicle on a dime, (c) so "soft" that they will prevent him from feeling any irregularities in the roadway over which he is traveling, and (d) so cheap that the manufacturer will give them to him. Although the latter objective will never be attained, the tire manufacturer can go a reasonably long way toward meeting any of the other three objectives but only at the expense of the remaining objectives. The possibility exists, therefore, that, as more progress is made in improving those characteristics of the pavement that contribute to adequate skid resistance, the tire manufacturer may become less concerned about that aspect of tire performance and concentrate on improving his product with respect to life, ride, or some other desirable quality. Such an occurrence would place the highway engineer in the position of "running to stand still." There are channels through which the highway engineer can make his concerns known to the tire industry and can keep informed concerning trends within that industry. For example, the American Society for Testing and Materials is currently establishing a committee on tires that will be concerned with many aspects of tire performance. The steering committee that has been making the necessary arrangements to establish this committee is well aware of the desirability of participation in the committee's activities by those having highway-oriented interests. It will be most unfortunate if the voice of the highway engineer is not heard in these councils, and it will be his own fault.

With respect to tires, it is not sufficient to ensure that the tire originally placed on a vehicle contributes its fair share to skid resistance. As tires are worn and tread depth is reduced, skid resistance effectiveness is also reduced. One recent study (16) has shown that an inordinately large percentage of vehicles involved in wet pavement accidents had badly worn tires on the rear wheels. If the highway engineer is to be charged, at least morally, with preventing or reducing accidents caused by inadequate skid resistance, he has a reasonable and real interest in the formulation of regulations that specify the wear level at which tires must be replaced, the establishment and operation of inspection procedures to evaluate compliance, and the enforcement of compliance. Some initial steps are slowly being taken in this direction, but much more action is required.

Brief mention was made earlier of the need for a public information program. Perhaps such a program is needed not only to generate public support for an anti-skid program but also to effect beneficial changes in driver behavior. For the past several years I have met, through the National Safety Council, a number of driver educators—some of whom have been actively engaged in this field for a number of years—who are not familiar with such fundamental traction phenomena as the fact that a vehicle sliding with its brakes locked cannot be steered. Some of them have only the grossest appreciation that a wet pavement has less skid resistance than a dry one and no feeling at all for the magnitude of the difference. If this is indicative of the state of knowledge among even a significant part of those whose business it is to understand and to teach the relationship between driver action and vehicle performance, it may be assumed that a vast portion of the driving public has little appreciation of what they ask of the roadway surface and the tires when they perform certain maneuvers and equally little understanding of what they are likely to get. Perhaps it is not too much to hope that in this era of mass communication appropriate informational and educational programs may be able to bring the average driver to such an appreciation and understanding. Such an accomplishment would surely reduce the problem of both the highway engineer and the tire manufacturer.

There is one final topic, perhaps not directly associated with anti-skid program management, with which those contemplating the institution of such a program should be concerned. In a number of states, civil actions for damages in one form or another have been brought against state highway departments by individuals injured in motor vehicle accidents in which some part or all of the allegation has to do with inadequate skid resistance. During recent years, elements of government have increasingly lost their immunity to suit, and it seems almost certain that in the near future it will be generally accepted that a citizen has the right to seek in the courts redress for losses allegedly incurred because of some improper action or inaction on the part of government.

A few suits of the type suggested have already been won by plaintiffs. One of the major problems faced by the plaintiff in such an action is the proof of what should be considered adequate skid resistance. Once adequate minimal skid numbers have been defined by state or other agencies, this burden will be removed from the plaintiff. He will then only have to allege and seek to prove that the skid number on the pavement involved in his accident was below the specified minimum at the time of his accident. The agency defending the suit will have to prove that the skid number of the pavement involved was indeed above that specified or that appropriate actions designed to remedy the inadequacy were being undertaken as expeditiously as possible. In view of the large number of accidents occurring annually, the increasing propensity of those involved to institute litigation, and the apparently increasing view of jurors that the injured should be in some way compensated, the potential for such actions against the state is staggering. Those engaged in the management of an anti-skid program should be aware of this potential and should begin plans at an early stage in the program for defense procedures to be used when such litigation occurs.

SUMMARY

The highway engineer or administrator has to involve himself in the following processes in establishing and managing an anti-skid program.

He must first determine that the problem is a real one that involves loss of life on the highway. He then must commit himself to the program, fully realizing that some of his actions will be economically or politically unpopular. He must sell this dedication to the public so that they will support the program, and in doing so he should attempt to educate the public.

He must determine the level of skid resistance that should be maintained on various elements of the highway system under his jurisdiction. He must decide, preferably in concert with other engineers and administrators from other communities, how the desirable or existing values of skid resistance shall be measured and evaluated. After having agreed on a technique for measurement, he must decide on a measurement program that will involve evaluation of specific locations at which there is other evidence of inadequate skid resistance, reconnaissance of considerable mileage of pavements to determine existing values of skid resistance and trends in performance that will permit scheduling of anticipated remedial treatments, and acceptance of newly constructed surfaces or remedial treatments to existing surfaces.

For each surface found to be inadequate he must choose the treatment to be applied: treatments that may be applied to the existing surface, application of a new surface, or major reconstruction involving improvements in geometric design. He must also be concerned with new construction to see that the geometric design is such as to place the least reasonable demand for skid resistance on the surface and that the materials selected and techniques employed in construction are such as to provide adequate and durable skid resistance. He must exercise care that in applying remedial treatments to eliminate an existing hazard he does not create a new one.

Finally, he should maintain an interest and awareness in those areas that are not under his immediate jurisdiction, particularly the areas of tire design and construction and driver education, and that may have an important influence on the demands placed on the roadway surface. He should exercise such influence as he can in these areas in order that the problems that he faces in his anti-skid program may be minimized.

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ASSEMBLY AND USE OF ACCIDENT DATA

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•THE use of accident records as a predictor of future accident occurrence and therefore as an indicator of necessary remedial action is an accepted approach in the highway safety field. The technique is familiar to safety engineers in all fields and has been one of the early traffic engineering techniques documented in the literature.

In many areas, unfortunately, the technique has not been used systematically. The extent of this situation was forcefully brought to the attention of many highway engineers when the then Bureau of Public Roads began its "spot improvement" program in 1964. Few state highway departments could produce a list of high accident locations, and almost none was able to demonstrate a systematic approach that maintained a routine flow of such information.

The significant words here are "a systematic approach" and "a routine flow of information." Although no standard pattern or procedure for such an approach exists, at least four key elements are essential in terms of an anti-skid program:

1. Accidents that result in more than a specified amount of damage, e.g., $100 or $200, should be reported;
2. A location information system that is capable of pinpointing accidents within $^1/_{10}$ mile (which almost always requires some form of field reference system) should be established;
3. Environmental information concerning each accident should be reported to permit identification of accidents caused by skidding; and
4. A storage and retrieval system that will provide quick and automatic notice of concentrations of skidding accidents at points or on sections should be established.

If these four prerequisites are met, the foundation exists for an anti-skid program that is soundly based on accident experience. The state of the art is such that these requirements can be met; and, although not all states have reached this point, there are enough examples to prove the practicality of the idea.

The spot improvement program, which has been reoriented toward the identification of high accident locations, has provided the impetus for up grading state performance in this area. The current Federal Highway Administration directive (1) on this subject says, in part:

> ...the following must be maintained on a continuing basis:
>
> (1) A field reference system for identifying the location of individual occurrences, such as reference posts, coordinates, etc.;
>
> (2) A traffic records system with the ability to correlate collision data with vehicle, driver, and highway data including the ability to correlate accident experience with existing geometric features and traffic characteristics at specific locations. An ultimate objective of the system should be to identify causative factors of highway collisions;
>
> (3) A procedure for identifying and reporting hazardous elements and locations based on accident analysis. This will involve analysis of actual accident experience at specific locations, analysis of accidents related to specific elements or geometric features throughout a route section, route system or area, and/or application of approved traffic forecasting procedures based on traffic characteristics....

Sponsored by Steering Committee for Workshop on Anti-Skid Program Management and presented at the workshop.

This approach was given considerable support by the publication of the Highway Safety Program Manual (6). That manual (Standard 9) provides, among other things, that there shall be "a systematically organized program to maintain continuing surveillance of the roadway network for potentially high accident locations."

The progress of the states in complying with the demands of the FHWA directive and Standard 9 has been mixed but generally satisfactory. As might be expected, some states have progressed further than others—partly because some were starting from a more advanced point. The several factors that go into a complete program of this nature begin with a field reference system; without such a system the problems of determining the highway location of accidents are greatly increased. Without good location information in fact, the whole system breaks down. In this area much has been accomplished by the states. Most states have installed mileposts, or reference markers, but not on every mile of road. In general states have installed mileposts on the Interstate System, and some have extended or will extend the system to all federal-aid primary routes. A few plan to reference heavily traveled federal-aid secondary routes, but not state has yet suggested that roads off the federal-aid or state systems be so treated.

A second factor is a records system to summarize data systematically and to correlate accidents with highway data. Every state has some sort of records system, but no real evaluation of these varied systems is available. At least 15 states are working with a consultant or with the National Highway Traffic Safety Administration to develop a complete records system. It would thus appear that a number of states see a need to improve their record systems and also that much needs to be done. Three years ago, 37 states responded to the question, Do you utilize state accident information for the detection of slippery pavement sections?; 19 said "Yes" and 18 said "No" (2). This suggests that, at least in 1967, the situation was not so good as it might have been.

That there is a correlation between accidents and slippery surfaces is clearly shown in Figure 1. The data are from an English survey of 150 miles of rural highways (3).

A third factor is the capability to identify, based on accident analysis, hazardous features and locations. Again, an accurate evaluation is impossible. It is believed that at least 80 percent of the states are currently able to use a method other than public complaint or engineering judgment to identify hazardous locations. The most commonly used methods to justify the term hazardous are about equally divided among three approaches: the number of accidents, a combination of number of accidents and the accident rate (called the number-rate method), and a statistical method that identifies a significantly high accident rate (called the quality control method). Although these three vary greatly in the degree of sophistication, any one is probably superior to the "brush-fire" approach that is sometimes used.

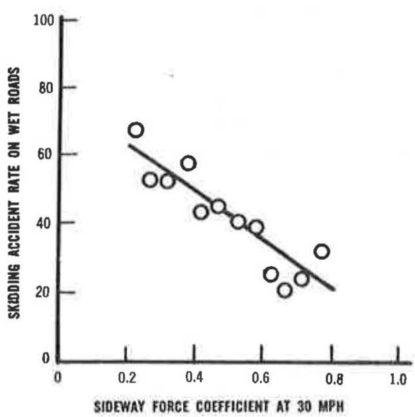

Figure 1. Relation between skidding accident rate and side force correlation.

In any event, the important point is that decisions are based on accident records, not on a preconceived notion that a particular section is slippery. The actual form of the accident information—the basic input of the system—is not important. The information may come from any of the several sources: driver reports, police reports, the sample data from a bi-level system, or reports of multidisciplinary teams. The usual statistical cautions must be kept in mind. Especially in the case of the sample from a bi-level system or reports of multidisciplinary teams, the size of the sample is important because the number of cases will probably be small. Using the maximum number of reports is the best way to avoid missing specific locations that might be overlooked or bypassed in a sampling process.

Another factor that must be considered is the recognized idea of "regression toward the mean." If a location has a high accident experience for a given period of time, the chances are that in a following period the experience will be lower. It thus becomes very important to determine that the apparently high experience is truly high. This means a sufficiently long time period with a large enough exposure and significant number of accidents to be meaningful.

This question of meaningful or statistically significant figures leads directly to another aspect of accident records and their application to an anti-skid program: the use of these records to measure results or to evaluate the program. The common technique used is the conventional "before-and-after" comparison. Here again, a number of caution signs must be heeded. As has just been suggested, both the before and the after experiences must be significant. A statistical tool applicable here is the "quality control" method, which is actually only a statistical device to help ensure that the before experience is high by a statistically significant amount—or at least is a reliable, meaningful figure whether high, low, or average.

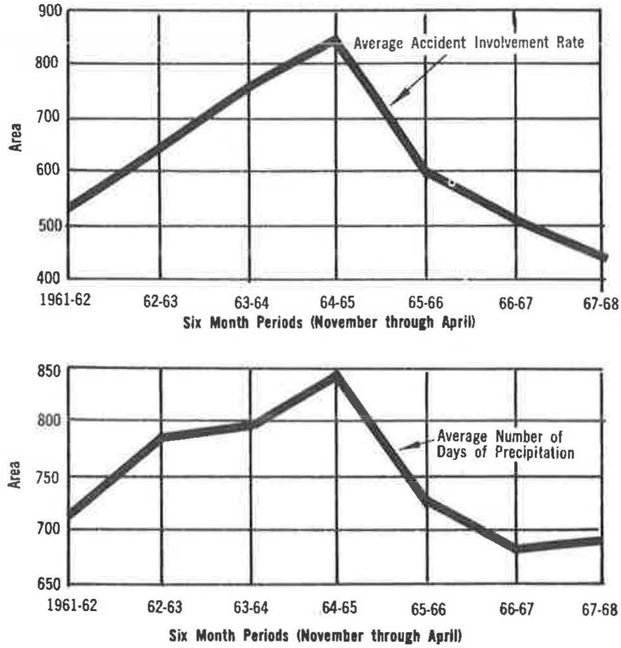

Figure 2. Relation between accident rate and precipitation.

Then, there is the question of the change that occurs, or perhaps does not occur. Are the before and after periods comparable in terms of time, weather, and traffic? This is particularly important when dealing with skidding accidents because comparing a rainy before period with a dry after period could easily show a benefit even if nothing else was changed.

The relation between precipitation and accidents is shown in Figure 2. The shape of the two curves is the significant factor. The top one shows average accident involvement rate, and the bottom one shows average number of days of precipitation. These data are based on a study of 48 busy intersections in Lexington, Kentucky, during a 6-year period (4).

Some experts in this area take a dim view of any before-and-after comparison unless a control section is used as a standard against which the test section can be measured. That is somewhat difficult, if not almost impossible, in the kinds of situations with which we are concerned. However, that would be a much more scientific method of evaluating results and is mentioned here not only as a desirable objective but also as a further emphasis on the fallibility of the conventional before-and-after study.

An example of the use of accident records to measure the effectiveness of anti-skid program efforts is the work of the California Division of Highways. The report by Beaton and others contains the following illuminating paragraph (5):

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

There were 30 wet days in the before period and approximately 15 wet days in the after period. Fifteen additional wet days were accumulated from the following year and the accidents on these days were added to the after period.

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

Those results are interesting—perphaps even exciting—and are important to other aspects of anti-skid programs. The important point in this discussion is the care taken by California to ensure significance and comparability in their figures.

In summary, we can say that the state of the art has advanced such that accident records can provide sound guidance to a state in its anti-skid program, and a number of very significant programs currently in operation. However, the overall performance level of the states is not yet up to its full potential. The composite picture is mixed, of course, with some states having far more sophisticated approaches than others. Because of the growing realization of the importance of accident data for the total highway program, all states are improving their levels of performance in this area.

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USE OF ACCIDENT DATA TO IDENTIFY WET-PAVEMENT LOCATIONS IN PENNSYLVANIA

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•IDENTIFICATION of wet-pavement accident locations is one of the specific uses of the accident data bank that has been in operation in Pennsylvania since January 1966. The responsibility for all statewide accident analysis was assigned to the Accident Analysis Section of the Bureau of Traffic Engineering.

Accident reports are received from state and local police and from involved motorists. The forms used by the state and local police are almost identical. The local police are furnished the forms by the commonwealth if they agree to supply a copy of each completed investigation. Drivers involved in accidents resulting in death, injury, or property damage of more than $100 are required to file reports. Reports of 293,000 accidents were analyzed during 1969.

The essential data for accidents that might have been affected by wet-pavement conditions are date, severity, location, weather, pavement condition, vehicle type, collision diagram, and description of the event. The value of all subsequent analysis depends on the quality and accuracy of the information provided by these reports.

It should be noted that information not subject to change, such as speed limit, traffic volume, roadway width, and type of pavement, is not included in accident reports. Those data are included in the road log that is in computer storage, and they can be matched against the accident data for any section of state highway that is being studied.

The commonwealth has also established a reference system that is primarily based on station markers for the state highway system and on mile markers for the Interstate system. Highways are identified by legislative route numbers, rather than by traffic routes, because the legislative route system covers all highways under state jurisdiction and is the index for road log and other records. Locations on other than state highways are identified by street name, township road name or number, and block face. The need for a more complete reference system on rural roads not under state jurisdiction is recognized. However, a study of reported accidents indicates that 94 percent of those accidents occurring on the state system can be adequately located for remedial purposes within the location framework now in use and that all but 2½ percent of the accidents on other roadways can be located adequately by route name or cross route. Achieving this coverage is based on the assumption that knowledge obtained from nearby residents or maintenance personnel in rural areas can pinpoint locations if the problems are serious enough to warrant a field study.

ANALYSIS

Reports received from police and operators are correlated and given sequential identifying numbers. All papers referring to a particular accident are stamped with the same number, which is used to cross-reference driver files. After the analysis is completed the papers are filed by number for future detailed reference. Because of this correlation reports can be grouped by county or city, which somewhat simplifies the location process.

As a first step in analysis, a team of about 15 people reads the reports to determine the location as accurately as possible. The team depends primarily on large-scale county maps, but they also use all available reference material such as logs, telephone books, and street indexes.

Sponsored by Steering Committee for Workshop on Anti-Skid Program Management and presented at the workshop.

Another group of about 15 analysts record the vital information for each accident for placement on magnetic tape. The analysts do not simply record the information given in the reports. They read the reports thoroughly and then form an opinion as to what actually occurred.

As a result of this analysis, information is coded that shows the following items pertinent to wet-pavement accidents: accident record number; location; date; weather; road surface condition; accident type; identification of offending vehicle by type, movement, and causation factors; identification of other vehicles involved; and severity of accident. For each accident this information is available in a broken-English abstract for electronic data processing.

DATA PROCESSING

More than 80 different programs have been written for analysis and tabulation of the data available from the accident data files. Three programs are of particular interest in an analysis of wet-weather accidents.

Cluster listings based on the criterion of 12 accidents or more within 1,000 ft during a 3-year period are prepared for the state highway system. The threshold figure obviously varies with the time period being studied, and a somewhat larger number must be used for a shorter period and must be developed with experience. The distance criterion is based on a "moving" 1,000 ft, not on successive segments. The information is printed on a single line for each location and shows the county, legislative route number, limiting stations, number injured, number killed, and number of accidents. This information is printed in sequential order by route number for ready reference and in descending order by number of accidents for priority action. Engineering personnel, who are part of the surveillance team in each district, are assigned to review all locations on the cluster lists, to determine the relationship between the highway environment and the reported accidents, and to suggest countermeasures. Obviously, some of the clusters will relate to wet-pavement accidents and will be treated accordingly.

Tabulations are also made of those sections of highway on which an apparently high proportion of accidents has occurred during wet weather. In those printouts the data are arranged by legislative route number and are subdivided by change in pavement type and abutting construction sections. Information is shown for the number of accidents, the rate per hundred million vehicle-miles of travel, the accidents per mile of road, and the percentage of accidents occurring on wet pavement. (These accidents are defined as those for which the accident report shows the weather as "rain" or the pavement as wet or both. Snow-covered or icy pavement is excluded.)

As part of a "search-by-hazard" program, another type of cluster report has been developed for accidents occurring on wet pavements. A minimum of five accidents must occur within a 3,000-ft segment of road during a 3-year period for the information to be listed. This information is printed on one line and is similar to that described in the previous paragraph.

FIELD STUDIES

The various printouts of wet-pavement accidents are studied, and a list of highway sections to be skid-tested is prepared for each engineering district. Two skid trailers are used in the statewide testing program, which includes tests on other sections reported to be slippery, retests, and research studies. Present procedure calls for any section with a skid number of less than 30 to be treated immediately and those with a skid number of between 30 and 40 to be placed on future programs for resurfacing or other corrective treatment.

In cooperation with the National Highway Traffic Safety Administration, the department has established accident investigation and surveillance teams of engineers and state police in each of the 11 engineering districts. Their work is divided into two phases, the first of which is in-depth accident investigations. Although some accidents that involve slippery pavement are investigated, the total number is not great enough to give wide coverage or statistically stable results. However, if slippery pavement is suspected to be a causal factor in a specific accident, that location will be skid-tested.

The second phase of the teams' work is involved with the review of all cluster locations, as indicated previously.

GENERAL COMMENTS

The program for correcting pavement surfaces with low skid numbers based on accident studies has not been in operation long enough to allow for after studies of accidents, although these are obviously desirable. It is anticipated that the 1971 accident records will provide a basis for this comparison.

As noted earlier, it is incumbent on testing personnel, pavement specialists, and automotive engineers to recognize that the quality of an analysis is dependent largely on the data provided. If more specific data are needed, changes will have to be made at the reporting level. Meanwhile, available information will provide for a program that uses all available skid-testing equipment and financial resources for corrective treatments.

SAFETY INSPECTION OF THE HIGHWAY AND THE VEHICLE

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•THERE can be no single solution to highway safety for accidents may be caused by any one or combination of five factors: society, highway user, highway facility, highway environment, and vehicle. The degree to which each factor reacts either alone or with other factors is not fully understood and bears continuing study if a desired level of safety on highways is to be achieved. In June 1967, the National Highway Safety Bureau (NHSB) issued the initial standards relating to 13 areas associated with highway safety; in November 1968, 3 additional standards were issued. These 16 standards identify and describe basic performance goals in the major problem areas of highway safety. Two additional standards—school bus safety and accident investigation—are currently being developed. There are several program areas to be considered in managing a state program directed to providing the safest possible highway system. Although some problem areas can be studied in independent efforts, they should be studied in interdisciplinary efforts to obtain the best possible results. For example, driver education programs will be more effective if the program material keeps pace with changes in highway design as well as changes in safety features of the automobile. The areas that are the subject of this paper, highway and vehicle inspection, are vital to the development and management of an effective anti-skid program.

SAFETY INSPECTION OF THE HIGHWAY

The states, through the Federal-Aid Highway Safety "Spot" Improvement Program, have completed the programming of approximately 21,600 improvements at high-accident locations on the federal-aid system. Although the exact description of these improvements were not reported, it is known that many involved corrective measures for improving pavement surface skid resistance. Where skid-resistance measurements are involved, the states have worked in many instances through the Federal Highway Administration in the development of skid trailers for measuring the level of skid resistance. The skid trailer has become the most widely used equipment for ascertaining the level of skid resistance of pavements.

The Department of Transportation's Instructional Memorandum of April 28, 1968, provided a major "push" to make highways skid safe. This memorandum stated the following:

> . . . the skid resistance qualities of questioned pavement surfaces shall be tested by operating thereon at a speed of 40 miles per hour with a two-wheel skid resistant trailer or equivalent device following the procedures outlined in ASTM E-274-65T, Tentative Method of Test for Skid Resistance of Highway Pavements Using aTwo-Wheel Trailer and the ASTM Technical Publication No. 366 - Measuring Road Surface Slipperiness . . . .
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> It also provides that the Bureau of Public Roads (FHWA) will participate in the resurfacing costs of maintaining state highway surface skid resistance at a skid number level of 0.35. Hence, the desire for the states to inspect and inventory their pavement surface is predicated to a degree by the willingness of FHWA to participate in the cost of corrective measures.

There are several methods for measuring pavement skid resistance. They may be grouped conveniently in four general classes, three of which involve a moving vehicle: (a) portable devices, (b) automobile stopping distance, (c) automobile deceleration, and

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(d) towed trailers. Over the years, the greatest improvements have occurred in the development of towed-trailer techniques. Only modest improvements have been achieved in the stopping-distance automobile method.

Briefly, the most widely known portable methods used in this country are the National Crushed Stone Association's (NCSA) bicycle wheel, the British portable tester, and the Penn State drag tester. The NCSA wheel is now being used to a lesser degree than the two testers.

The NCSA method (1) consists essentially of determining the degrees of rotation of a bicycle-sized wheel with the tire sliding against the pavement surface after being started by an initital constant rotational force. The greater is the angular rotation of the wheel, the more slippery is the surface.

The British portable tester was used widely for field measurements in the early 1960s, but it is now being used cautiously because of a better understanding of its limitations. It is a dynamic, pendulum-impact type of tester and essentially consists of a rubber slider fixed to a shoe that is attached to an arm that rotates about a fixed point. When the arm and attached shoe are raised to a fixed level and then released, the pendulum swings unrestrained except when in contact with the surface under test. The height (indicating energy loss) that the shoe travels after sliding over the surface is an indication of the surface frictional resistance. This method is standardized as ASTM Designation E 303.

The Penn State drag tester is a small test unit that uses a test shoe of the type used with the British portable tester. The tester is pushed by the operator over the test surface at a normal walking speed of 2 to 3 mph. An indication of the surface resistance is obtained by reading the dial that measures the "drag force" caused by the test shoe when the tester is pushed at a constant rate across the surface.

In each of the three portable methods, it is necessary to block traffic while the test is being conducted. Also, if other than dry tests are desired, it is necessary to wet the surface prior to each test. There have been efforts to correlate these test methods with one another as well as with other methods. Although the results have not been completely successful, they are reported in the referenced literature.

The automobile stopping distance method (2) is one of the three moving-vehicle methods. It consists essentially of driving a test car at the desired test speed over the surface to be tested, locking the brakes, and allowing the car to slide to rest. The sliding distance is measured, and the coefficient of friction is calculated. This test method approximates real-world situations and is most frequently used by state highway patrolmen to spot-check accident locations. The method is currently being reviewed and readied for acceptance as an ASTM standard.

The second moving-vehicle method involves the measurement of momentary deceleration of a test car. This method (3) is usually conducted with a decelerometer mounted on the floor of the test vehicle. The vehicle is brought to slightly above the desired test speed, and the brakes are momentarily applied hard enough to cause a short, quick skid. The deceleration of the test vehicle during the short skid is a measure of skid resistance. This method, like the stopping-distance method, is not so convenient to conduct because it requires the close control of traffic and a watering truck for wetting the pavement area to be tested. Both methods, however, have been used in statewide inventory work.

The most widely used method for measuring pavement skid resistance involves a moving vehicle with a trailer in tow. This method is used in 29 states. There are four states in which the task of building a trailer or purchasing one from one of the several commercial sources is being considered. The development of the towed-trailer method has been evolving since about 1920, and the two-wheel trailer is now an ASTM standard method (ASTM Designation E 274). In this method either a one- or two-wheeled trailer may be used. The test is conducted by towing the trailer at a predetermined test speed over the section of pavement to be tested, locking the wheel(s), and measuring the force required to "drag" the trailer at a constant speed. The surface skid resistance (or skid number) is calculated from the known or measured forces acting on the trailer during the test. The advantage of this test method is that the test can be conducted in moving traffic because of the trailer's self-contained watering system.

In summary, the inspection of a particular highway facility in regard to an anti-skid program involves all features of the facility, but emphasis should be placed on the roadway pavement characteristics (level of friction). Characteristics related to the ability of the motorist to perform necessary maneuvers are of paramount importance (e.g., the level of pavement friction needed by a driver to negotiate a turning maneuver or an emergency stop).

Also, in such a program, minimum levels of skid resistance will need to be established and will of necessity be influenced by the type of facility and its anticipated use. In addition to these facets of the program, enforcement techniques such as "wet-weather" speed limits and safety inspection of automobiles will need to be considered.

SAFETY INSPECTION OF THE VEHICLE

In addition to providing a continuing inventory and inspection of facilities, an anti-skid management program can be effective in identifying and seeking corrective measures for motor vehicles. This can be most effectively accomplished by working with and through periodic motor vehicle inspection systems.

The first subject of the initial 13 standards issued by the National Highway Safety Bureau is periodic motor vehicle inspection. The introduction to this standard states the following:

> Until recently there was very little firm evidence to support the reasonable supposition that state inspection systems contribute to highway safety. This deficiency has now been overcome, at least, in part. Recent research demonstrates significant differences in state motor vehicle accident death rates associated with inspection programs. Although much more specific information is needed, especially with respect to the extent to which various kinds of inspection contribute to the overall results, it is clear that the inspection of motor vehicles by the states has an important place in highway safety.

The purpose of this standard is "...to increase, through periodic vehicle inspection, the likelihood that every vehicle operated on the public highways is properly equipped and is being maintained in reasonably safe working order...."

Motor vehicle inspection in the United States began with a voluntary program in Massachusetts in 1926 (4). Although two other states participated in voluntary programs, Massachusetts, Maryland, and Pennsylvania enacted legislation in 1929 requiring inspection of all motor vehicles. At present, 31 states, the District of Columbia, and Puerto Rico require inspection of nearly all vehicles. These programs vary in manner of operation, as well as details of inspection, and have exhibited varying degrees of success. For example, 2 states and the District of Columbia operate under entirely state-owned and -operated systems, 27 use the state-appointed and -supervised system, and 2 (Florida and South Carolina) have combination systems. Of the 19 states that do not have an inspection law for all vehicles, 8 have spot-check programs that are administered on the highways by specially trained officers on a random basis, 6 have programs directed to certain vehicles or authorize inspections to be conducted on a local level, and 5 have not adopted any form of inspection. In Tennessee, where local option is in effect, five cities have an inspection requirement. Three of these are large metropolitan areas; two, Chattanooga and Memphis, have very fine municipally owned and operated inspection lanes.

Not only does the manner of system operation differ among states, but there is also a difference of opinion as to what should be inspected. In general, however, all systems require the inspection of brakes and lights. The extent to which other items, such as steering mechanisms, suspension, and exhaust systems, are inspected varies considerably.

The greatest deterrent to the states complying with standard 1 of the 1966 Safety Act lies in the belief that there is insufficient research data to show that periodic motor vehicle inspection significantly reduces traffic fatalities. The lack of information regarding accident causation as related to vehicle condition may be attributed in part to inadequate accident records. This inadequacy was recognized at the time the initial highway safety standards were promulgated, as they included standard 10 on traffic records. Thirty-two states, including Tennessee, are now studying, improving, and developing their records-keeping system and are including a uniform reporting form. Along with the development of adequate records systems, there is an urgent need to acquire on-site information concerning the influence of vehicle condition on accident causation. Data being collected in programs that are training accident investigation teams of the type under way at Georgia Institute of Technology will be useful in identifying this relationship. In this regard, McCutcheon and Sherman concluded in their study that "...it was found that the mechanical condition of a vehicle population is substantially improved as the frequency of inspections increases and that the number of defects per rejected vehicle decreases as the frequency of inspection decreases" (5).

Another indication of vehicle condition is inferred from the safety defect recall campaigns that are required by the 1966 Safety Act. For example, during a 3-month period (January 1, 1970, to March 31, 1970) manufacturers reported that 223,234 foreign and domestic vehicles were recalled for reasons varying from wheels with improper welds to incorrect steering shafts that could become disconnected and cause loss of steering (6). Another example of defected vehicles on highways is illustrated in the work of the Bureau of Motor Carrier Safety. In 1969, the Bureau inspected "... 397 buses operated by 107 different interstate motor carriers of passengers. Forty-seven (11.8 percent) of these were ordered out of service until corrections essential for safe operation have been effected" (7). There is evidence that periodic motor vehicle inspection will help ensure that better maintained automobiles will operate on the streets and highways.

There are two general approaches to the development of an inspection system: the voluntary and the compulsory. The voluntary system is one in which there is no formal legislation or, at the most, only limited legislation. The compulsory system, on the other hand, requires legislation. There are 19 states that do not have a law requiring periodic inspection of all registered vehicles on a statewide basis. Of these 19, 14 either require that only certain vehicles be inspected, such as school buses, or may have a "voluntary" inspection program. A study of the literature did not reveal data for a comparison of citizens' attitudes toward voluntary and compulsory systems. I believe, however, that voluntary systems can be made effective by appropriate controls. Generally, voluntary systems are thought of in relation to diagnostic centers or one's own garage mechanic. The basic types of compulsory systems are either state owned and operated or state appointed and privately operated. The nature of the inspection procedure and the type of facility can range from inspections provided by the owner-operator to those at well-equipped, state-owned and -operated inspection lanes. Each type of system seems to have its unique advantages. The most frequently discussed systems are given in Table 1.

Diagnostic clinics (or an individual's private garage) represent the ultimate in the truly voluntary type of system. They have been popular in Europe for several years, having been made available mostly through automobile clubs. They have become generally available in this country during the last 10 years. About the time of the approval of the Highway Safety Act, clinics were enjoying rapid development. More recently,

TABLE 1

TYPES OF INSPECTION SYSTEMS

Type	Ownership and Operation	Facility
Compulsory	Owned and operated by state or city	Permanently located inspection lanes Portable facilities Fixed facility and roving inspectors
	Appointed by state and operated privately	Garages and service stations
	Operated privately under state contract	Permanently located inspection lanes
Voluntary		Diagnostic clinics Random spot-check Trial substitute

however, they have not enjoyed such prosperity because of their high equipment cost and general lack of public interest in their service. The centers are designed to test many items, including those that are not necessarily safety related. Although several nationwide organizations, such as Sears Roebuck, Montgomery Ward, J. C. Penney, and major oil companies, have diagnostic centers, the most comprehensive one seems to be the rather modern clinic that is operated by the St. Louis Automobile Club. It has equipment to test a great number of items, and many of these tests can be made with the vehicle operating at speeds under load to simulate actual driving conditions. Diagnostic clinics have the potential of providing a greater amount of information, accurately obtained, than do smaller facilities, but the cost of services related only to safety inspection may be too high for widespread acceptance.

Although established by legislation, the random spot-check system is somewhat voluntary in that the motorist does not have to report to a specific location for the inspection. This system consists of roadside checks usually performed by the state's highway patrol. It is conducted at various times and at random locations throughout the state. It is reported (8) that in California about 15 percent of all vehicles are checked annually and about 62 percent of those fail. In the California system, those vehicles that pass inspection receive a sticker. The owners of those vehicles that fail must have the defect corrected and the vehicle reinspected. During the inspection, the officer checks windshield and side-window views, spray-on window tints, muffler condition, lights, tires, and other items that have been judged as safety related. At present, eight states (California, Michigan, Minnesota, North Dakota, Ohio, Oregon, and Washington) use this system with slight variation. The effectiveness of the spot-check approach is closely related to the level of enforcement and the penalty imposed.

The trial substitute system is also a partially voluntary system, but it does require limited legislation. This system (9) is unique and relatively untried; it authorizes the owner to inspect the vehicle himself or to have someone do it for him. The inspection is required at regular intervals, after all reportable traffic mishaps, and at the time of the purchase of a new or used vehicle. Each vehicle owner is provided inspection guidelines and two types of vehicle inspection certification forms. One provides for a minimum level of inspection, whereas the other encourages the owner to conduct a more extensive check. The monitoring of the system is accomplished by regular enforcement channels in addition to spot-checking. The advantages of this system are reported to be that the motorist will be better informed of the operating condition of his vehicle; he will not be subject to the potentially unscrupulous actions of service station attendants or garage attendants or garage mechanics; his cost is less than it would be with most systems; and his repairs may be done as needed. The immediately apparent disadvantage is that the great majority of owners who inspect their own vehicles are not skilled and may believe that a vehicle component is safe when it is not.

Although voluntary systems are intriguing to the individual motorist, these systems are not generally satisfactory from the viewpoint of uniformity of inspection and ensured compliance. This is not to say, however, that such systems cannot be made effective.

Compulsory systems are the most desirable type for ensuring uniformity and compliance. Although there are several approaches to such systems, the state-appointed and privately operated system is in most widespread use. This type of system is currently operating in 29 states, two of which (Florida and South Carolina) have combination systems. In this system, the state defines the program and then licenses private garages, service stations, and other groups, such as automobile dealers and fleet operators to perform the inspection. In general, inspection facilities are not elaborate, but, usually, an area is reserved for the performance of the work. The equipment cost depends on the items to be inspected. The state charges an authorization fee—usually \$25—to discourage casual and "fly-by-night" operators from obtaining a license. The inspection fee of the vehicle owner is set by the state and frequently ranges between \$2 and \$5. The state's portion of the fee is generally about 50 cents.

The principal advantage of the state-appointed and privately operated system is the low initial cost to the state. The vehicle owner has the advantage of usually having an inspection station nearby; frequently, his regular service station will be an authorized inspection station. A major disadvantage lies in the difficulty in providing uniformity in inspection. For example, because the inspectors are not generally trained by the state and because the equipment will not be equally maintained, there will be a different level of inspection among and between inspection stations. There is also a major disadvantage to the inspection station operator because he does not receive adequate compensation for services performed. Also, if new items, such as exhaust emission, are to be checked, the equipment cost to small operators may be prohibitive.

State-owned and -operated systems currently exist only in Delaware, New Jersey, and the District of Columbia. Florida and South Carolina, as earlier reported, have dual systems whereby they combine the state operated with the privately operated. The state-owned and -operated system is quite similar to the municipally operated systems in Chattanooga and Memphis, Tennessee. Although not well documented, cost may be the reason for so few state-owned systems. Once the system is in operation, however, inspection fees can be established at a level sufficiently adequate to support the program. New Jersey has about 40 stations that serve approximately 3,000,000 registered vehicles, whereas Delaware has only four inspection facilities. The Delaware stations are located such that there is at least one station within 35 miles of a vehicle owner. The distribution and location of inspection facilities may be governed by vehicle density and population movement. For example, in heavily populated areas it may be wise to locate the facility near a major shopping center so that the inspection can be made while drivers are on a shopping or work trip. In any case, adequate planning is needed to ensure public acceptance. Also, in this type of system, it may be desirable to have permanent locations in remote areas that would be staffed only on a periodic basis. For example, research sponsored by the National Highway Safety Bureau has resulted in the development of a portable, truck-mounted facility that serves the remote areas of a state, not unlike the early American "rolling store" idea.

The advantages of the state-owned and -operated system are that it separates inspection from repair, provides uniformity in inspection because of uniform equipment and personnel training, is easy to monitor and update, is convenient for training personnel, and will probably provide the revenue for supervision and enforcement. The greatest disadvantage lies in its initial cost. This is particularly true if it is not instituted on an incremental basis.

The state-contracted system is another method of providing the attributes of a state-owned and -operated system by contracting with a private company to design and operate the system. Basically, a private firm would negotiate a contract with a state to finance, design, erect, equip, operate, and maintain the necessary facilities required to provide the state with a comprehensive vehicle inspection program. The private firm would assist the state in preparing public-relation materials to secure acceptance and response to the system. The state, in turn, would have to pass the appropriate enabling legislation to authorize the program and to ensure a degree of continuity for a period of about 10 years, which is roughly considered to be the amortization period for the land and building.

Several of the reported advantages of such a system are that it offers an inspection program established by an independent contractor who has no vested interest; it offers the motoring public a program of greatest value received per dollar spent; and it offers the engineering expertise and experience of private firms. The greatest disadvantage seems to be in the difficulty in ensuring support by the state and its citizens for a minimum period of 10 years. If public acceptance is not ensured, the system could become a major liability.

In the design of a vehicle inspection system, the most critical factor with regard to the relationship of vehicle condition and accident rate is the vehicle features to be inspected. Because there are insufficient data to indicate this relationship, considerable judgment must be exercised in the determination of items to be inspected. For example, there is considerable difference of opinion as to the influence of front-end alignment. In view of the lack of firm data, it is appropriate to classify inspection items into those that the driver must correct (i.e., the vehicle is rejected during inspection) and those that he is advised to correct. It is desirable to reevaluate periodically the items in these classifications on the basis of records and experience and to make appropriate changes. An indication of the frequency that particular items cause rejection was reported by Coverdale and Colpitts (10) on the basis of information reported by the New Jersey Division of Motor Vehicles for a 2-month period in 1963 (Tables 2 and 3). Headlights were rejected most frequently in vehicles less than 10 years old. Brakes and all other lights were the next most frequently rejected items. These data also were supported by those given in Table 4 from the voluntary national vehicle safety check that was reported in 1963 (11). In addition, the rejection frequency was born out in a study of the Memphis system but was not supported by data from the Knoxville system. The Knoxville data show that wheel-alignment defects had the highest rate of rejection followed by lighting systems. The results of a mechanical factor investigation of 409 fatal single-vehicle traffic accidents in California revealed that the braking system was the most commonly observed mechanical defect. Steering system defects, which accounted for 26 percent of all defects, were next (12).

TABLE 2

VEHICLES INSPECTED DURING 1963 VEHICLE INSPECTION IN NEW JERSEY

Vehicle Age (year)	Inspected	Approved	Rejected	
			Number	Percent
Under 1	76,368	57,616	18,752	24.6
1 to 5	214,876	129,561	85,315	39.7
6 to 10	145,801	74,218	75,583	50.5
Over 10	484,817	281,791	203,026	41.9

TABLE 3

ITEMS REJECTED DURING 1963 VEHICLE INSPECTION IN NEW JERSEY

Item	Vehicles by Age Having Deficient Items (percent)				
	Under 1 Year	1 to 5 Years	6 to 10 Years	Over 10 Years	Total
Headlights	16.9	19.6	20.10	20.6	19.4
All other lights	4.9	12.8	2.12	23.7	15.1
Brakes	2.7	10.0	17.00	23.5	12.1
Steering operation	.4	3.3	7.70	11.5	5.0
Steering alignment	2.4	4.1	6.70	6.8	4.9
Directional signals	1.1	2.9	4.80	4.3	3.3
Windshield wipers	0.2	1.4	4.50	6.0	2.6

TABLE 4

ITEMS REJECTED DURING 1963 NATIONAL VEHICLE SAFETY CHECK

Item	Items Rejected on Cars		Items Rejected on Trucks		Items Rejected on Cars and Trucks	
	Number	Percent	Number	Percent	Number	Percent
Rear lights	90,960	18.8	12,892	17.7	103,852	18.7
Front lights	69,120	14.3	8,931	12.3	78,051	14.0
Brakes	49,366	10.2	7,016	9.6	56,382	10.1
Rear turn signals	43,929	9.1	6,138	8.4	50,067	9.0
Front turn signals	42,182	8.7	6,047	8.3	48,229	8.7
Stop lights	35,112	7.3	9,834	13.5	44,946	8.1
Exhaust system	36,562	7.6	3,627	5.0	40,189	7.2
Tires	33,715	7.0	5,140	7.1	38,855	7.0
Windshield wipers	19,197	4.1	3,128	4.3	23,045	4.2
Steering	18,276	3.8	1,893	2.6	20,169	3.6
Glass	15,499	3.2	3,224	4.4	18,723	3.4
Horn	13,974	2.9	2,822	3.9	16,796	3.0
Windshield washers	9,091	1.9	622	.9	9,713	1.8
Rearview mirrors	5,051	1.1	1,479	2.0	6,530	1.2
Total	482,754	100.0	72,793	100.0	555,547	100.0

Note: Total vehicles checked--3,448,976.

The broad categories of items that should be inspected for any total system are lighting and electrical systems; steering alignment and suspension; tires, wheels, and rims; body glazing and sheet metal; exhaust and fuel systems; and brakes (13). For a defined set of conditions, the malfunctioning of one or more vehicle components within any one of the broad categories listed could cause an accident. For an anti-skid management program, however, the most significant category is vehicle tires because of their contribution to the skidding phenomenon. The motor vehicle inspection system should include tires among items to be inspected. Data obtained in a midwestern state indicated that about one-third of the cars involved in 631 accidents had defective tires based on a minimum tread depth of $^2/_{32}$ in. Of this group, 22 percent had tread depths less than the established minimum. In some few instances, states that have a vehicle inspection system are beginning to inspect tires. Initially, inspectors are advising the owner of defects and inadequate tread depth and are rejecting his car in extreme cases only. Retreaded tires and off-the-road tires are coming under greater scrutiny by the National Highway Traffic Safety Administration. Recently, an advisory circular was sent to all states calling attention to the undesirable use of off-the-road tires on freeways. Because the sidewalls of such tires are now marked, the tires can be spotted during the regular inspection cycle. The NHTSA held a public meeting in January 1971 to discuss a proposed amendment to standard 109 requiring tire manufacturers to label passenger car tires with information on the number of times they can be retreaded. Tires make an important contribution to the skidding phenomenon; therefore, a successful anti-skid program will need to be closely coordinated with a vehicle inspection system to ensure adequate inspection and corrective measures.

SUMMARY

Although accident causation is not clearly delineated, it is believed that the number of skidding accidents is significantly great to warrant continued study. In this regard, the inspection of highway characteristics, as well as an inventory of pavements as to their level of skid resistance, must be undertaken by each state. Also, it is of great importance that the periodic motor vehicle inspection program be reviewed in context with the objectives of the anti-skid programs to achieve and maintain an appropriate relationship between the two efforts. An anti-skid program is needed in each state because it can be an effective adjunct to a state's total effort in making highways safe for motorists.

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A SYNTHESIS OF CASE LAW JURISPRUDENCE RELATING TO WET-WEATHER HIGHWAY CONDITIONS

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• THE increase in litigation caused by the derogation of sovereign immunity and by other factors has resulted in an increasing interest on the part of highway administrators in the sovereign's duties and consequent liability where those duties are not met. Legal liability for accidents occurring on icy and wet highways is an area that the public has been especially concerned with in recent years.

The extant case law on the subject has established three central areas and one subarea in the jurisprudence of maintenance liability. These areas are (a) compliance with general duties in order to escape liability; (b) damages resulting from noncompliance (negligence); (c) contributory negligence as a bar to recovery; and (d) advisory signing as a technique in meeting general duties.

There are several reasons for the past failure of contributory negligence theories. The chief reason has been the belief in social compensation engendered in such theories as that of the "deep pocket." Briefly, this theory states that restitution should be made by the element most capable of bearing the burden of loss. In cases where public agencies or agents are concerned that element is the public, who through the state should compensate an injured victim. Of course, flagrant negligence on the part of the claimant will operate to bar recovery even when there is negligence on the part of the state.

The public sector, in this age of growing inflation, population pressures, and public cognizance, is near the brink of bankruptcy. The state no longer has the limitless revenues necessary to meet all demands made upon it. Choices will have to be made. In this atmosphere, I think it can be hypothesized that contributory negligence will have an increasing vitality. Each and every case will be studied thoroughly to determine not only any negligence on the part of the state but also any negligence on the part of the claimant. Administrative rules and regulations, ordinances and statutes, and advisory practices will place more and more responsibility on vehicle operators to avoid negligent operation of their vehicles or risk the possibility of not recovering damages even when there is proven negligence on the state's part.

The use of advisory signing, mainly speed limits, is an area that has not yet been fully developed. The establishment of a doctrine will gradually appear in this area as the body of case law on contributory negligence grows. Certainly, where advisory signing is used, an immediate onus will be placed on motorists to comply for fear of being held contributorily negligent. However, the coin has two sides. Where advisory signing is used, an awareness on the part of the state of latent dangerous conditions is inferred. This will place the duty of special care on the state to ensure the safety of these areas.

Where recovery has been allowed for accidents that occur on icy, snow-covered or rain-slicked highways, it has usually been because these conditions have created a defect or obstruction. In other words, recovery is absolutely based on more than just the presence of slick conditions. Often such decisions seem wholly unfair to those charged with the duty of maintaining the road network because of poor roads, inadequate personnel, and sparse funds.

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On the other hand, where recovery has not been allowed because of the absence of some committed or omitted operation that created a danger to motorists, the decision seems unfair to the individual. Frequently, there is little that a motorist can do to avoid an accident resulting from a slick spot on a highway, and, in the absence of liability imposed on the state, the injured innocent must bear the brunt of the loss.

LIABILITY ON THE PART OF THE STATE

Let us first look at those cases that have discussed the allowance of recovery based on negligence of the state. (Because our research showed that few important cases occurred before 1956, the cases referenced in this paper date from 1956.) An early New York case (Babcock v. State, 191 N.Y.S.2d 783, N.Y., 1959) involved a claimant who had been injured when his car skidded on an ice patch. The evidence at the trial was held determinative of the state's negligence in allowing water to collect and form ice on the highway, which created a hazard to motorists. The court concluded that the state had actual notice of the condition because the maintenance foreman knew of the icing. Water frequently ran over the road at the locus of this accident, and it was common knowledge that cold weather would result in freezing conditions. Even more indicative of judicial attitude was the statement, "No adequate flare or sign was placed to warn a driver of the dangerous condition that existed..." The court further held that contributory negligence was not in issue here because testimony established that the claimant had been driving at 35 mph, the weather had been clear, and the road had been dry except at the locus of the accident where there had been ice and slush.

Another case involving improper drainage occurred in the Washington, D.C., area (Jennings v. U.S., 207 F.Supp. 143, D.C., Md., 1962). A question arose as to liability on the theory of nuisance in connection with icy road conditions that had formed as a result of insufficient drainage facilities. The court studied the testimony and found that the drainage facilities had been inadequate, that frequent users of the highway had been aware of the problem, and that the agency responsible for maintenance of the highway should have been aware of the situation. The court further found that the existence of an insufficient drainage system had created a condition that was a nuisance, that the patch of ice on the highway was attributable to a long insufficiency of drainage facilities, and that the patch of ice was the effective cause of the injuries in the complaint. It was further found that the government could have abated the nuisance and was therefore liable.

In both of these cases a common thread can be detected. Both notice and acts of omission were present in each instance. In one case it was failure to warn by signing; in the other case it was failure to reform a defective condition. The total impact of these cases is brought home in a third case (Stern v. State, 224 N.Y.S. 2d 126, N.Y., 1962) in which the claimant had been driving at 40 mph when he noticed a posted speed of 30 mph. As he attempted to slow his vehicle, it began to skid. It was raining and a collision occurred. The court laid down basic principles early in the decision. Because of their universality, they are quoted as follows:

> It has been universally held by the courts of this state that the State of New York owes a duty to the public to keep its highways in a reasonably safe condition for travel and that a duty rests upon the state to maintain warning signs on a state highway if circumstances presented reasonably demand....Ordinarily a defendant is not liable for conditions due solely to the weather, but where the highway because of some existing condition becomes more slippery than the usual highway when wet, and is rendered dangerous for the traveling public, the state may become liable to those injured thereby.

In light of these principles, the court queried whether this highway became abnormally slippery when wet. State troopers testified, as did other witnesses, that there had been oil spots on the highway and that a slippery condition had existed that remained until the oil film was washed away. The court predicated recovery on this basis, saying, "...there is an established fair preponderance of evidence that the macadam portion of the highway in question becomes more slippery than normal pavement when wet and that such conditions existed prior to the accident." The state, concluded the court, should have been aware of the condition and was negligent because it had not removed the hazard or warned travelers of its presence. (We will return to this case later in our discussion of contributory negligence.)

The next case involved an unusual accident (City of South Bend v. Fink, 219 N.E. 2d 441, Ind., 1966). The claim was for damages resulting from death by drowning that had allegedly occurred when an automobile left the road and proceeded into a river. The road had been barricaded and used as a recreational sledding area. Prior to this accident, the barricades had been removed, and the claimant contended that this had indicated the street was safe for travel. However, the claimant also contended the street had been neither salted nor sanded, nor had there been removal of curb accumulations of snow and ice. Neither barricades nor guardrails had been present along the descent of the street to prevent a car from leaving the road and crashing into the river. The principle of law upon which a remedy is granted or denied in such a case may be stated as follows:

> ...reasonable care must be exercised to keep a street in reasonably safe condition for travel. But there is no liability for injuries caused by deposits in the street due to the natural accumulation of snow and ice. Reasonable diligence under such conditions has been interpreted to mean timely notice and an opportunity to remove the accumulation.

In this situation, the court found that the city had actually encouraged the creation and continuance of a dangerous condition on the hill and that the condition had not been merely the result of a natural accumulation of snow and ice. In such a case, an instruction using the words "active vigilance" as a requisite of the city is not confusing or ambiguous to the jury. There is no evidence supporting such a notice on the part of the city, and the terms "active vigilance" and "reasonable diligence" considered with all other evidence is not reversible error. Similar terms to these have been used in many cases, too numerous to cite here, in referring to knowledge of a defect in a vehicular way.

The final case in this section (Freeport Transport, Inc. v. Commonwealth, Dept. of Hwys., 408 S.W. 2d 193, Ky., 1966) involved a truck that had skidded off the highway. A claim filed with the Board of Claims against the Department of Highways was denied on the grounds that (a) the defect had not been created through negligence and (b) the department had had no notice of the condition. The Circuit Court upheld the Board of Claims. In the Court of Appeals, the evidence revealed that a bituminous sealing had begun to flake off shortly after repair, which exposed the primer coat and allowed the surface to become slippery when wet. Eight months after the initial flaking, the accident occurred. There was no contradiction by the defendants that a dangerous condition had existed. In light of this, the court found the Board of Claims had tacitly assumed the existence of an intermittently hazardous condition. Notice as a basis of liability was discussed, and the court said that, although no actual notice was present, "...circumstances may have been such that knowledge was imputed or presumed." The time factor was an important element in this determination. For 8 or 9 years prior to the patching job, no accident had occurred at this spot; however, in the 8 months following it, there were seven accidents. Finally, both the Board of Claims and the lower court had emphasized the existence of a hazard when the road was wet, which seemed to indicate a duty to inspect only when the road was dry. The court felt that the latent nature of the defect would make it less easily discoverable, but this was just one of the circumstances to be considered in determining whether the defect should have been discovered over a long period of time. Another consideration was the type of traffic and design of the highway involved. The court then concluded that, under the circumstances, a dangerous, latent defect had been shown to exist for a sufficient period such that the department should have had notice of it. The failure to discover and correct it constituted negligence. There was a dissent by Justice Montgomery:

> The effect of the holding is that it is made the duty of the Department of Highways to keep under constant inspection, wet weather or dry, every mile of the many thousands of miles in this state. This includes toll, interstate, federal, primary, secondary, and rural highways. When it

> rains how often must the department inspect for slick spots on the road up Chicken Gizzard Ridge and over to Possum Trot?
>
> This decision holds that the department is responsible for any accident attributable to any so-called defective road condition that has existed for eight months because it is presumed that the department will by then have notice of the condition. The availability of funds for such purpose is not considered....

NO LIABILITY ON THE PART OF THE STATE

As was indicated in the preceding section, the ordinary standard of care to be used by the state in carrying out its duties is that of reasonable anticipation. In certain circumstances the state has been found to have fallen short of that standard. In most cases, however, a policy that is established as a result of discretionary procedures and that is followed as setup is sufficient to meet the standard.

Such a situation arose in a New York case involving the scheduling of maintenance patrols (Wheeler v. State, 156 N.Y.S. 2d 660, N.Y., 1956). In that case there was an action for injuries that had been sustained when an automobile skidded on ice and crashed into a bridge abutment. The road was clear on the afternoon of the accident, but there had been a light snow the previous day. The ice had apparently formed when shoulder snow melted and ran across the highway. The locus was known as a possible ice-hazard spot. The highway on which the accident occurred was regularly patrolled by a maintenance crew, but 5 days prior to the accident the crew had been placed on summer schedule and Saturday routine patrols had been stopped. There was a reasonable belief that the winter patrol schedule would have revealed the ice. The court found that the state was not liable. There was no evidence of actual notice nor had sufficient time passed for constructive notice to be made. The main question was whether the state should have anticipated a snowfall on April 8 or 9. Testimony indicated that sanding this late in the season would have been unusual. The court found the evidence too meager to determine whether reasonable care would have demanded continuance of winter scheduling through April.

It is also well established that, to prove liability on the part of the state, it is necessary to show that negligence is the proximate cause of an accident (Edwards v. State, 159 N.Y.S. 2d 589, N.Y., 1957; see also Gladstone v. State, 256 N.Y.S. 2d 493, N.Y., 1965). Accordingly, a condition that would not be hazardous in clear weather and icy conditions in general along great stretches of roadway even though ice is present at a possible defective drainage site are not enough to prove proximate cause.

It is also not enough for a claimant to show that a highway is slippery because of rain. It is common knowledge, or so it seems from these cases, that macadam becomes slippery when wet; hence, there must be proof of some defect, obstruction, or obligation within the state's control to prove negligence. Negligence is not to be inferred from a car skidding; nor is there a duty upon the state to construct and maintain shoulders such that they can be used for general travel. The standard here is a reasonably safe condition for use by a prudent driver traveling at a reasonable speed in an emergency (Eckerlin v. State, 184 N.Y.S. 2d 778, N.Y., 1959).

In an interesting case (Commonwealth, Department of Highways v. Brown, 346 S.W. 2d 24, Ky., 1961), suit was brought because of an accident caused solely by an icy condition of a curve on a highway maintained by the Department of Highways. The department had been fully aware of the condition for 36 hours prior to the accident. However, the department in no way contributed to the condition; it simply had not cleared the highway of snow and ice at the locus. The ultimate question was whether it was the duty of the department to remove snow and ice from the highway or to give warning of dangerous conditions caused by the natural accumulation of snow and ice. The court interpreted the statute governing the case as not placing such an affirmative duty upon the state. This was so even though the state had for years assumed responsibility for cleaning the highways on a regular basis. That activity was considered a gratuity. (In connection with this case, dissenting opinion of Justice Montgomery in Commonwealth v. General & Excess Insurance Co., 355 S.W. 2d 695, Ky., 1962, should be read and compared to his earlier dissent in the Freeport case.)

It is generally held that the state is not an insurer but is responsible for maintaining reasonably safe highways. If this general duty is carried out with reasonable diligence by the state, an accident resulting from natural causes will not make the state liable. Such factors were present in a case where an automobile that had been traveling at a speed of approximately 25 mph during a snowstorm was involved in a collision (Dodd v. State, 223 N.Y.S. 2d 32, N.Y., 1962). There had been no defect in the highway, and the salting crew had arrived within moments of the accident. The court found that the accident had occurred in broad daylight when merely driving on the road created a hazard of unusual risk.

The next case (Tetreault v. State, 269 N.Y.S. 2d 812, N.Y., 1966) presents several interesting considerations that merit detailed discussion. It involved a tractor-trailer that had skidded off the highway at the crest of a hill during a snowstorm. The road was slushy and deteriorating. The claim was based on negligent construction, maintenance, and repair of the highway and on failure to erect adequate warning signs of dangerous conditions. Although the Bill of Particulars covered negligence due to snow, slush, and ice on an unsalted and unsanded highway, the court allowed an investigation of the highway banking because there was no prejudice, surprise, or bad faith. Referring to this investigation, the court found that "...the highway was crowned in the center and banked to the west.... There is no expert proof that such was improper construction." In order for the claim to succeed, then, negligence in maintenance had to be proved. The evidence at the trial led to these conclusions: The claimant's rig had been traveling at about 15 mph at skid; there had been approximately 2 in. of snow on the ground; and the maintenance crew in charge of sanding and salting had been notified of the conditions and, at the time of the accident, was proceeding to the hill with a truckload of salt. The court said:

> The fact that there was snow on this road, that the road was slippery, and, that the subject vehicle skidded and jack-knifed, does not establish negligence against the state. Although the state is under a duty to maintain its highways in a reasonably safe condition for travel, it is not an insurer of the safety of its highways. Therefore, before we can fix the state's liability, we must determine whether or not it met the standard of reasonable care on the maintenance of its highway under the circumstances prevailing. Certainly, the state cannot be held to a standard of care which requires it to maintain a 24-hour watch over thousands of miles of highway during the winter months. Such is particularly true in the mountain regions of Northern New York, where many times a driver must weigh the necessity of reaching his destination against the readily perceivable dangers of continuing his journey.

The court further found that the road conditions had not existed an inordinate length of time and that the state employees reacted with celerity.

The following case (Christo v. Dotson, 155 S.E. 2d 571, W. Va., 1967) involved an interpretation of a West Virginia statute imposing liability for a roadway being out of repair. It established the general rule that, regarding municipal corporations, snow and ice alone do not constitute a defect. The court cited a Kansas statute and similar interpretation as a basis. The rule, according to the court, was stated as such: "In order to establish liability based upon a defect in a street or highway of such nature for the street to be considered out of repair, there must be an accumulation of snow and ice amounting to an obstruction...." The court also cited an earlier case (Boyland v. City of Parkersburg, 90 S.E. 347) that indicated that the word repair as used in the statute included an obstruction on the highway as well as defects in it and that, although ice and snow may render a street out of repair, there must be an accumulation amounting to an obstruction before it comes within the purview of the statute. The court concluded:

> Even if there were some ice on the bridge where this accident occurred it appears questionable whether or not it was the proximate cause of this accident without which there could be no recovery. However, there is nothing in the evidence contained in the record of this case to indicate that even if there were some ice on the bridge in question there was any accumulation of such magnitude as to create an obstruction and to come within the meaning of the statute as being "out of repair."

The next case (Gambino v. State, 280 N.Y.S. 2d 91, N.Y., 1967) involved a car that had skidded on the downside of a hill, struck a tree, veered to the opposite side of the road, and collided with a concrete culvert. A rain, snow, and sleet storm had preceded the accident, and the road was wet and icy in spots. The claimant maintained that improper construction and maintenance of the highway were responsible for the accident. An examination of the record by the court revealed that (a) there was no testimony that the driver's view of the hill contributed to the accident; (b) there was no evidence that holes or ditches on either side of the road contributed to the accident; and (c) a state exhibit revealed score marks in a herringbone pattern on the pavement. The score marks were described as being a system, or a pattern, of grooves embedded or impressed in the surface of the concrete pavement in a herringbone pattern. It was further stated that the apex of this pattern was at the centerline of the pavement, and the ends pointed downgrade approximately 45 deg with the centerline at that angle. There was no dispute that the purpose of the pattern was to increase the friction of the surface and to produce added traction. It is interesting that the claimant's expert referred to the system as "ineffectual and possibly adverse", apparently without contest by the state. But the court found no evidence in the record that this condition had contributed in any way to the accident. The court concluded that, under the circumstances, to find negligence here would be to place an "impossible burden" upon the state and be tantamount to making the state an insurer. In the words of the court, "Sudden storms, as with frost conditions, are part of nature, dependent upon the season of the year...." The court also stated:

> ...It had a duty to construct and maintain its highways in a reasonably safe condition, in accordance with the terrain encountered and traffic conditions to be reasonably apprehended. But even so, a certain risk was unavoidable. Roads cannot always be straight and level, and curves with descending grades are always potentially dangerous. A highway may be said to be reasonably safe when people who exercise ordinary care can and do travel over it safely.

CONTRIBUTORY NEGLIGENCE

Although the doctrine of contributory negligence has not had much acceptance, it is always considered in liability cases because it is an affirmative defense.

In a New York case (Scheelde v. State, 160 N.Y.S. 2d 686, N.Y., 1957), there was a claim for damage resulting from a skidding accident that occurred on slippery pavement. The claimant testified that he had been driving at approximately 35 mph, that the road, weather, and visibility had been clear as he approached the locus of the accident, but that he had not seen the condition complained of. However, evidence of weather bureau observations on that day established that about $\frac{1}{10}$ in. of snow had fallen between 11:00 a.m. and 3:30 p.m. on the day of the accident, and this was enough to rebut testimony of clear weather. In utilizing this evidence the court cited an ancient Chinese proverb that said, "The palest ink is better than the most retentive memory." The court further inferred from the record that the accident occurred during a snowfall that caused the conditions complained of. The alleged negligence on the part of the state was for failure to sand. However, there was no proof of actual notice, and the time lapse was too short to impute constructive notice. There was some evidence of previous accidents that could give rise to notice of a dangerous condition of which warning should have been given, but the evidence was vague and indefinite and was not sufficient to prove negligence as the proximate cause of the accident. In discussing contributory negligence, the court said:

> Under the weather conditions prevailing at the time, "the presumption of safety" of public highways "will not serve as an excuse for blind indifference to consequences,"...of, as insurance against the risks inherent in automobile travel in the winter,...and, the failure of the driver to slow down the speed of his automobile and to proceed with caution is evidence of negligence which is the sole proximate cause of the accident, and, is imputable to the claimant....

In the Stern case, discussed previously, it was found that the evidence militated against a finding of negligence on the part of the driver. The most relevant factors were (a) the tires on the car were new; (b) the driver had never driven the road in adverse conditions; and (c) there was only one relevant sign, a 30-mph speed sign. The claimant had cautiously decelerated the vehicle by tapping the brakes lightly, and he did not apply his brakes with force until the vehicle had crossed into the opposite lane. There was no evidence of failure to control the car, and the claimant succeeded in his suit. The court's procedure in this case is very interesting; it leads to the conclusion that bald tires as well as reckless speed or loss of control can be equally responsible for defeating a plaintiff's claim.

It has further been held that in a two-vehicle collision, contributory negligence on the part of both drivers will operate to bar recovery regardless of the condition of the highway (Christo v. Dotson, 155 S.E. 2d 571, W. Va., 1967).

The manner in which a vehicle is operated is a primary consideration in the examination for contributory negligence. Factors such as the distance traveled from the first skidding, the ability to retain the pavement, and leaving the pavement, returning, and leaving on the opposite side of the roadway all act to impute doubt on the manner in which a vehicle has been operated (Gambino v. State, 280 N.Y.S. 2d 91, N.Y., 1967).

In the following case (Frehafer v. State, 301 N.Y.S. 2d 156, N.Y., 1969), the claimant was driving up an incline of a three-lane road (two lanes proceeding southerly—the claimant's direction). While passing vehicles, he went over the crest of the hill and observed that the road narrowed to two lanes. He applied his brakes, skidded on the wet pavement, fishtailed to the opposite side of the road, and collided with an oncoming vehicle. Although the court found state negligence in designing and signing the highway at the place of the accident, which operated as the proximate cause of the accident, the court also found that a prudent driver would have had his car sufficiently under control on the crest of a hill in anticipation of possible hazards. The speed limit was 50 mph, which was the claimant's speed. The court concluded by saying:

> Upon the present record, it cannot be said that the sign which indicated that the claimant's outside lane would narrow rather than his driving lane created a trap because there is no evidence that claimant observed the sign and, therefore, the sign, as such, could not be the proximate cause of the happening of the accident. However, as claimant approached the crest of the hill there was a "no passing" sign and before that there was the sign indicating that the road narrowed. While those signs have been inadequate to warn the claimant that his lane would shift to the right, nevertheless they were adequate to warn of a hazardous driving condition ahead and out of sight as one approached the crest of the hill. Under such circumstances a reasonable and prudent person would have proceeded at less than the posted rate of speed or would have been prepared to brake his car so as not to lose control when he reached the hazardous situation on what was described as a wet and slippery road.

ADVISORY SIGNING

There has been little discussion of advisory speed signing in the case law. Other signs, such as warning signs, directional signs, and lane-change signs, have been discussed more frequently. As a practical guide, it is sufficient to say that signing should be used at all locations where a hazardous condition has formed and has been brought to the attention of the state. It has, in fact, been stated that the duty to warn the public of a dangerous condition rests upon the party who has the duty to maintain the highway. The courts have held it to be negligent to permit a dangerous condition to exist without adequate warning (Citron v. County of Nassau, 268 N.Y.S. 2d 909, N.Y., 1965). By implication, it is fair to say that advisory speed signs may be included therein.

A California case (Johnston v. County of Yolo, 79 Cal. Rptr. 33, Cal., 1969) included expert testimony designed to show that a road jog constituted a hidden danger that should have been revelaed by warning signs and reflectorized "paddle" markers. Sections 830.4 and 830.8 establish immunity for a condition consisting solely of a failure to post regulatory or warning signs and signals. The second sentence of section 830.8 qualifies this immunity where there has been a failure to post warning of a hidden danger. The county charges error in the jury instructions covering this phase of the case. The court held that the lower court did not err in giving an instruction in the following form:

> A public street or highway is not in dangerous condition as that term has been defined solely because of the fact that the entity did not provide a speed restriction sign. Physical conditions such as width, curvature, grade and surface conditions, or any other condition readily apparent to a driver, in the absence of other factors, do not require special downward speed zoning, because the basic speed law under the instruction which I have already given is sufficient regulation as to such conditions.

The court further examined county justification for the restriction as an expression of Vehicle Code section 22350, the basic speed law that forms part of a comprehensive system of substantive rules covering public tort liability for dangerous conditions of public property. The system's rules of liability are qualified by its rules of immunity—sections 830.4 and 830.8 comprehensively describe the relationship of traffic sign posting to the liability and immunity rules. The court also felt Vehicle Code section 22358.5 served to discourage speed limit signs made needless by obvious road conditions.

Although that case is restricted to somewhat limited circumstances, it does aid in giving some flavor of judicial reasoning where advisory speed signing is involved.

INVESTIGATION OF INJURY-PRODUCING AUTOMOBILE ACCIDENTS AT THE HIGHWAY RESEARCH INSTITUTE

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•THE investigation of automobile accidents that result in injury has been a long-standing research program at the Highway Safety Research Institute of the University of Michigan. The research program was begun in 1961 under the direction of Donald F. Huelke. The initial purpose of this program was to determine the cause of death of accident victims and the mechanisms by which these fatal injuries were sustained. This was accomplished by investigating fatal automobile accidents in Washtenaw County, Michigan. The premise for this project was that accurate, detailed data of this type were not readily available because police officers investigating accidents do not have the time or the training to obtain the necessary information.

The program has now evolved to one in which not only fatal accidents but also injury-producing accidents are studied. The vehicle population studied comprises automobiles up to 2 years of age, light trucks and vans up to 3 years of age, and large trucks and tractor-trailer units up to 10 years of age. The purpose of the program is to determine the type and severity of injuries resulting from traffic accidents and to relate these to the design features of the interior structures of the vehicles involved. Many of the data obtained are used to evaluate the performance characteristics of vehicle safety features such as energy-absorbing steering columns, high penetration resistant windshields, padded instrument panels, and seat-belt restraints. Emphasis is placed on the injuries produced versus the injuries prevented. Accident causation is also studied because of the important part it plays in establishing the kinematics of the vehicle occupants and the manner in which injuries are produced or prevented.

The accident investigations involve cooperation with police departments, hospital medical personnel, towing service operators, ambulance attendants, and the vehicle occupants themselves. These investigations include medical reports of the injuries, police reports of the accidents, inspection of the vehicle to assess damage and to determine the interior contact points of the occupants, measurement of the vehicles and the accident location, and photography of the accident scene and the exterior and interior of the vehicles. Cases studies are prepared and presented periodically to the sponsor, the Automobile Manufacturers Association.

The current program now has more than 450 cases in the file and 900 to 1,000 cases planned for the entire 10-year project.

Sponsored by Steering Committee for Workshop on Anti-Skid Program Management and presented at the workshop.

HIGH-SPEED SKID TESTING AT UNIVERSITY OF TENNESSEE

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•MOST skid tests carried out to date have been at modest vehicle speeds, i. e., 30 to 40 mph. Skid coefficients for higher speeds have been extrapolated from the results of these tests. The work of the Tennessee Highway Research Program (THRP) and other organizations has pointed to the questionable accuracy of this procedure.

Because the Interstate System of highways has provided the public with opportunities to travel at sustained speeds of 70 mph or higher, some knowledge of friction coefficients is needed for those speeds. A high-speed skid trailer built by the THRP and capable of operation in the 75- to 80-mph range is now being used to obtain high-speed skid data. These data are being used in two ways:

1. To attempt to find a minimum acceptable high-speed skid coefficient through a study of wet-pavement accidents and
2. To correlate high-speed friction coefficients with those usually measured (at lower speeds) to determine if high-speed data are truly necessary in order to classify a pavement as slippery.

To date, 80 test locations have been chosen from which accident data have been collected for the calendar year 1967. Although results are too sketchy to be statistically valid, indications are that correlations between wet-pavement accidents and high-speed coefficients are similar to those obtained by using data obtained at lower speeds. Additional work is planned in this area.

Work correlating pavement slipperiness rankings (using coefficients measured at 40 mph) with similar rankings for higher speed data has produced encouraging results. Use of the Spearman Rank Correlation Coefficient and the Student t-test (at the 1 percent significance level) indicates that there is no difference between highway slipperiness ratings made with low-speed data and those made with higher speed data. These results will be amplified when complete data are available.

On the basis of work done thus far, it may well be that more costly, delicate, and hazardous high-speed skid tests will not be necessary to assess a pavement's likelihood of causing a wet-pavement accident. Certainly, results to date are increasing our knowledge in this area.

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CALIBRATION OF BRAKE FORCE TRAILERS AND SKID-RESISTANCE PHOTO-INTERPRETATION

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•THE slope calibration method, used by the Department of Highways, Ontario, for the past several years, has several desirable features. It allows for the redistribution of the trailer load during braking; the precision is of the order of ±1.0 skid number point; and routine periodic calibrations can be performed in minutes, and workshop facilities are not needed. In this method, the wheel torque is measured while the trailer wheel is in a skid-resisting state and the tire patch point is relocated. In other skid-trailer calibration methods that use torque measurements, the skid numbers are calculated for the wheel-hitch load distribution of a nonresisting tire. The relocation of the tire patch point can produce significant differences in the skid test results obtained by the torque measuring technique for different tires (1).

Three pairs of wooden wedges are used. A metal plate faced with slip-resisting abrasive paper is placed on top of the wedges. The wedges are about a foot long and have a slope of 20 to 100, 30 to 100, and 40 to 100.

The wheel load W_x of a given trailer for slope x is obtained on a weigh scale with the wheels resting on the wedges. N_x is the calculated wheel load component normal to slope x.

For calibration checks, the braked trailer is placed on each pair of wedges in turn, and the wheel torque is recorded. The trace on the electronic recorder multiplied by the ratio of the ordinary wheel load N_0 to the component normal to the slope is skid number X for the wedge slope x.

The demands on the services of a skid trailer are heavy, and it is often difficult to avoid delays when the pavement skid resistance of actual or suspected potential accident locations has to be evaluated. The Department of Highways is using photo-interpretation of skid resistance (2), and it appears that, even in its present state of development, the method helps to reduce the number of problem locations by positively identifying pavement textures in the lower, middle, and higher ranges of skid resistance.

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INTERACTION OF VEHICLE AND ROAD SURFACE

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•THE origins of skid-initiated accidents cannot be studied effectively without considering the interaction of the automobile, the highway, and the driver and their relationship to the total environment. The complexity of the system of interaction shown in Figure 1 is impossible to describe. As in almost all problems that involve a large number of variables, simplification and synthesis are necessary to avoid confusion. The interaction of driver, vehicle, and roadway has not been successfully synthesized; historically it has been fragmentized. One group of researchers has considered the driver; private industry has concentrated on the vehicle; and highway engineers have been concerned primarily with the roadway. Because vehicle characteristics are susceptible to and have undergone rapid change compared to roadway conditions, there exists today a mismatch between many vehicles using the roadways and the roadway system. Because the capabilities of drivers vary so widely, there is also a mismatch between the capabilities of many drivers and the functions they are required to perform to drive safely. Figure 2 shows how a deterioration in any one of the three major factors can produce such a mismatch (27). Elimination of these mismatches should be a primary goal of any anti-skid program.

DRIVER REQUIREMENTS

Three distinct maneuvers—acceleration, deceleration, and cornering—and two combinations of these, acceleration and deceleration while cornering, are required to drive a vehicle. The development of friction between the vehicle tires and road surface is necessary to accomplish these maneuvers. Newton's First Law of Motion is, "Every body perseveres in its state of rest or of uniform motion in a right line, unless it is compelled to change that state by forces impressed thereon." The forces developed by friction between the tire and the road surface are the "impressed" forces that produce acceleration, deceleration, and cornering.

The classic method of defining driver demand for friction has been to observe driver behavior in the real driving environment. A number of research projects were conducted between the late 1930s and the mid-1950s, but relatively little work has been done since that time. Consequently, our present design policies have changed little since 1954. Because vehicle capabilities and the demand for vehicle high performance have increased, the codes and policies formulated on the basis of the early research are still surprisingly adequate; exceptions to this may be stated, but they are relatively few.

Deceleration

A vehicle's deceleration requirement (in g's) is equal "numerically" to the friction requirement. Figure 3 shows this numerical equality. The understanding that these concepts are completely different while numerically equal is important to eliminate communication problems between those who prefer to talk about friction requirements and those who best relate to acceleration levels.

Sponsored by Steering Committee for Workshop on Anti-Skid Program Management and presented at the workshop.

An excellent summary of the work that has been done on friction requirements was developed by Farber (1). Wilson (2) reported in 1940 that the maximum average deceleration from 70 mph was approximately 0.7 g. This compares favorably to the maximum deceleration achievable by contemporary vehicles on dry pavement as reported by Tignor (3). This high level of maximum achievable deceleration is in sharp contrast to the actual deceleration demanded by most drivers. Studies performed by Crawford (4) and May (5) report an average friction coefficient demand of the 50th percentile driving group of 0.4 to 0.5 g for decelerations at 50 mph. A more recent study was conducted by Kummer and Meyer (6) using selected drivers both in business-section traffic and in highway traffic. They reported very few brake applications exceeding friction level demands of 0.35. This number represents the peak deceleration, however, and the average values would be substantially lower than the average values observed by May. A comparison of emergency "locked-wheel" deceleration with controlled deceleration is shown in Figure 4. The controlled deceleration pattern observed by Spurr (7, 8) results in low initial deceleration but a significantly higher value near the end of the stopping period. This is the reverse of the pattern of deceleration we would expect in a locked-wheel stop (Fig. 4).

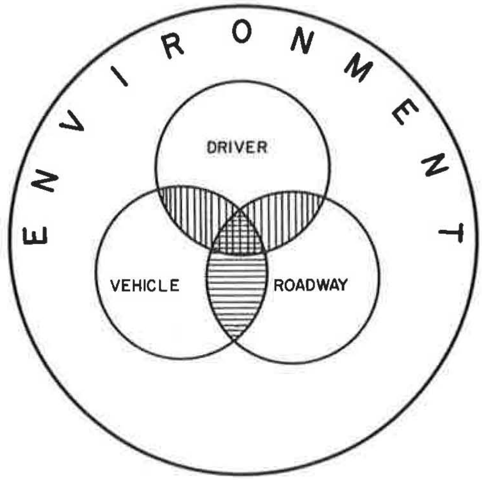

Figure 1. Interaction of driver, vehicle, and roadway.

Most of the recent studies of deceleration have been conducted by using selected drivers of test vehicles and are based on the assumption that a representative cross section of drivers is achieved and that the driver is acting as he would normally. Undoubtedly, economic considerations have played a large part in dictating this approach. The most desirable approach, from a statistical viewpoint, in determining driver behavior is to observe "real" drivers under "real" highway conditions. A project of this sort is now being undertaken at the Franklin Institute (1).

Acceleration

Of the three basic maneuvers, acceleration is the one that has drawn the least attention from highway engineers and researchers. Perhaps this is because almost all

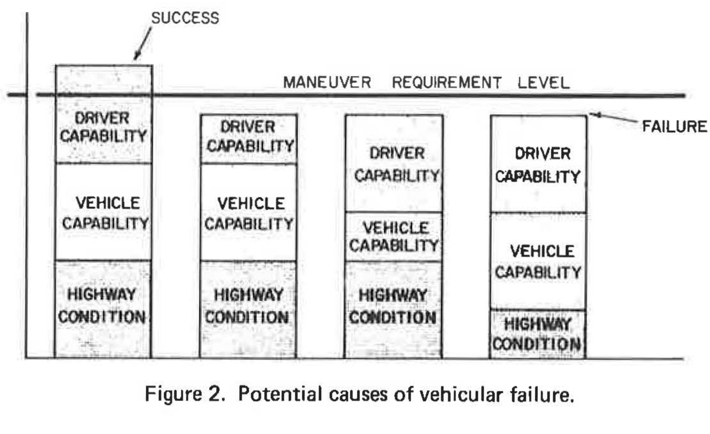

Figure 2. Potential causes of vehicular failure.

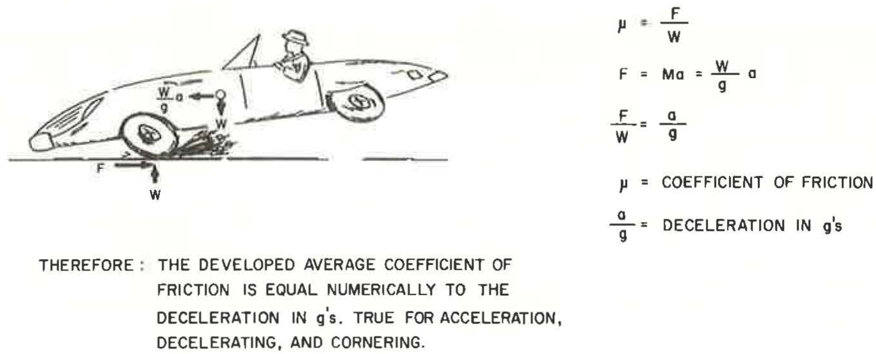

Figure 3. Numerical equality of coefficient of friction and deceleration.

emergency maneuvers, with the notable exception of the passing maneuver, do not require acceleration. Few drivers desire, or try, to achieve the maximum acceleration level of which their vehicle is capable.

A study of the passing maneuver has been reported by Weaver and Glennon (9). From the information presented, average accelerations can usually be estimated to be less than 0.05 g. Therefore, the simple acceleration maneuver does not seem particularly critical when compared to the higher demand for braking and cornering friction. It may become critical, however, when combined with the cornering maneuvers necessary to pass a slower vehicle. Design standards for length of acceleration lanes on freeways assume that the vehicle is capable of an average acceleration of 0.2 g, also considerably below the friction potential of most pavements in a wet condition.

Cornering

The classic study of vehicle demand for friction during the cornering maneuver was conducted by Taragin (10) in 1954. Taragin observed the speed of thousands of vehicles on rural roads. The horizontal curves included in his study ranged in curvature from 3 to 29 deg (1,910-ft radius to 198-ft radius). He found a high correlation between speed

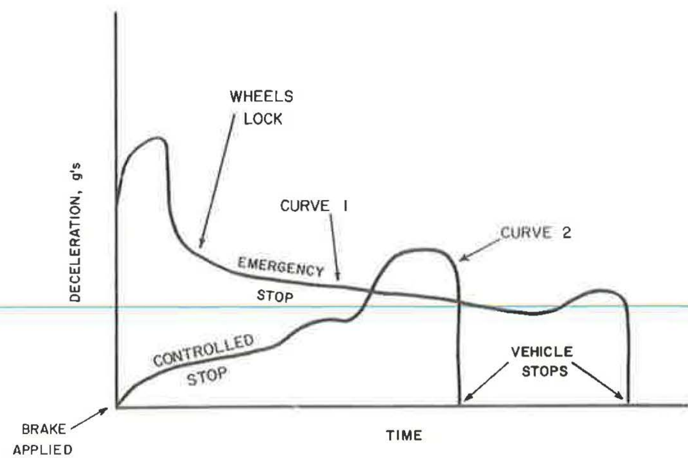

Figure 4. Deceleration patterns for controlled and emergency stops.

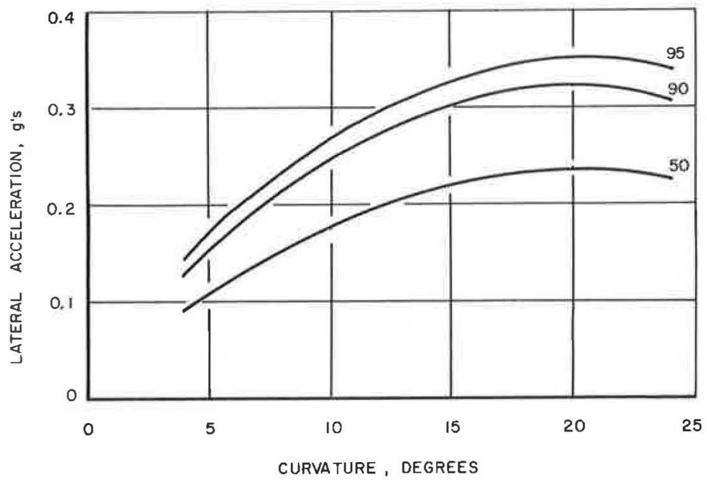

Figure 5. Taragin's relationship between lateral acceleration and curvature.

and degree of curvature. He developed a relationship between curvature and cornering force expressed in g's by using the classic relationship for lateral acceleration as a function of friction and superelevation, $F + E = V^2/15R$ (Fig. 5). The maximum lateral acceleration, or lateral friction demand, of 0.32 g for the 95th percentile driver occurred at a curvature of 20 deg. For the longer radius curves, 2 to 6 deg, the 95th percentile demand ranged from 0.1 to 0.2 g. The reference to 95th percentile means that 95 percent of the drivers observed made a smaller demand on lateral friction. In a recent study (11), the relationship between lateral acceleration and speed reported by Taragin was substantiated. Further substantiation is offered in other publications (12, 13). All of these studies, except the one by Ritchie (11), used the classic point-mass equation to calculate the friction demand and are, therefore, subject to the invalid assumption that the driver is traversing the same arc defined by the roadway.

Combinations

The demand for friction in combinations such as decelerating and cornering and accelerating and cornering has not been studied in the field, although Taragin observed that vehicle speed was relatively constant during cornering maneuvers. These combinations are scheduled to be studied by Farber at the Franklin Institute (1). The phenomenon of friction capabilities of a tire in the cornering-braking and cornering-tractive modes has been studied extensively in the laboratory (14). Highly sophisticated measuring equipment developed by the National Aeronautics and Space Administration, University of Michigan, Stevens Institute, and others is currently being used to study these combinations of tire operating conditions in the field. The Mobile Tire Tester at the University of Michigan is currently providing data for studies of the National Cooperative Research Program and the National Bureau of Standards. These studies should provide insight to the field observations of driver demand during these combination maneuvers.

ROADWAY GEOMETRIC REQUIREMENTS

Because of the difficulty in determining maximum friction requirements for vehicles, current geometric standards are not based on known factors of safety. They are based by necessity on the way people have been observed to drive. The knowledge of how the highway user's demand for friction relates to the total friction available is not well defined. The fact that a number of drivers each year are exceeding the friction supply is documented by the fatalities resulting from skid-initiated accidents. This fact in itself is not sufficient to indict our existing codes, but it does require us to continually re-evaluate the codes to ensure that they are consistent with contemporary demand.

Potential Problem Areas

A current study at the Texas Transportation Institute is devoted to defining the phenomenon of vehicle-pavement interaction during the cornering maneuver. Early results seem to indicate that the use of the classic point-mass equation in developing policies for highway curve design can be a fairly good approximation, provided it is coupled with accurate estimates of available lateral friction. The primary limitation of current curve-design policies is that the safety factor is directly dependent on an assumed available level of friction. Available friction varies widely on many pavements with vehicle speed and surface water depth as shown in Figure 6 (22). The other limitation of the point-mass equation is that it does not take into account the decrease in net available friction with shifts in the wheel load. The wheel load shift on vehicles approaching the critical skidding condition during the cornering maneuver can be as much as 50 percent (e.g., all wheel loads are 1,000 lb as a 4,000-lb vehicle moves through a tangent section of highway; however, during the cornering maneuver, the outside vertical wheel forces may go up to 1,500 lb, while the inside vertical wheel forces are decreasing to 500 lb per tire). Figure 7 shows that the resulting available friction decreases as wheel load increases. The result of load shift is a net lowering of available lateral friction.

Nondesign Maneuvers—Recent studies by Weaver and Glennon (15) have shown that the assumption that the driver traverses the arc defined by the highway lane is a tenuous one. Early data from this project, which concentrates on the observation of highway users, show that the radius of curvature driven by the vehicle may vary widely from the radius of curvature of the lane centerline and, consequently, that the instantaneous demand for lateral friction may be significantly greater than the average demand computed by using the lane curvature. Further verification of this is given by Kummer and Meyer (6) who state that vehicles occasionally demand as much as a 0.2 lateral coefficient when driving on straight tangent sections.

The passing maneuver is not treated by present curve-design standards. Figure 8 shows that a passing vehicle on a two-lane road must go through a minimum of four distinct curves. If the vehicle is on a horizontally curved, superelevated section of highway, two of these curves are against the superelevation; i.e., superelevation is reducing the potential for developing cornering forces rather than increasing them. A further decrease in the available cornering friction is produced by the tractive force necessary for acceleration.

Deterioration of Friction—Another potential problem in our geometric standards is the way in which the friction supply level is evaluated. This evaluation has been based

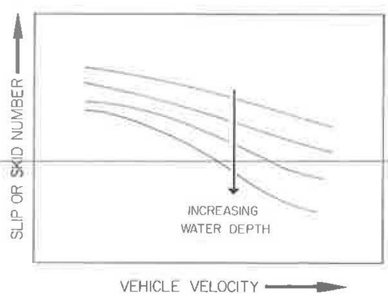

Figure 6. Variation of friction with water depth.

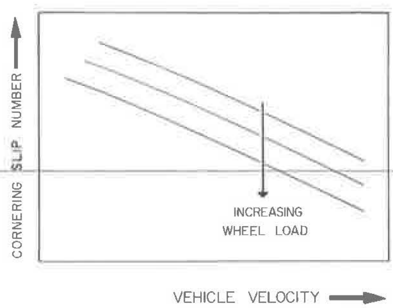

Figure 7. Variation of friction with wheel load.

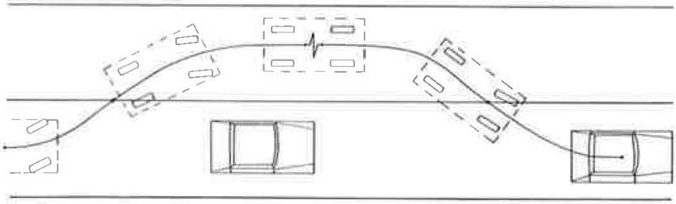

Figure 8. Curves traversed during passing.

in the past almost entirely on ASTM trailer locked-wheel skid numbers. The skid number of a particular pavement is not a constant but varies with a number of factors. The major factors are vehicle speed, surface moisture conditions, and polishing of the pavement under traffic conditions. The locked-wheel skid number at a speed of 40 mph determined by ASTM trailers has been widely used as indicative of available friction. For most pavements it is actually indicative of available friction in the locked-wheel mode at that particular time at a trailer speed of 40 mph. For many pavements, the skid number is highly dependent on the testing speed, and at higher speeds the ASTM trailer skid tester may actually give misleading results because the trailer's internal watering system does not adequately wet the surface before the test tire encounters it. For this reason, much of the skid number data determined at high speeds (more than 40 mph) may be biased. That is, skid numbers determined at 60 mph by using the ASTM trailer internal watering system may be higher than the skid number determined by using an external water system. One example of an external system is a tank truck depositing water on the pavement in advance of the skid tester. Another example is rain.

FACTORS INFLUENCING FRICTION FORCES

Vehicle

Brakes—When vehicle brakes are activated, the wheel rotation is slowed. Because the wheel-tire unit moves at a slower speed then does the vehicle, slip occurs at the tire-pavement interface. In addition, a force resisting the motion of the vehicle develops at the interface. This force, F, which is dependent on the tire, road surface, and lubrication is directly proportional to the braking coefficient, μ, which is defined as $\mu = W/F$, where W is the vertical force and F is the longitudinal force. This proportionality is true if W is held constant.

The braking coefficient varies with slip as shown by a typical curve in Figure 9. At 0 percent slip, the free-rolling case, the longitudinal force is zero, whereas at some small percentage of slip the maximum μ is generated. At 100 percent slip, the coefficient of friction is less than the maximum. A curve of similar shape will describe the lateral friction coefficient.

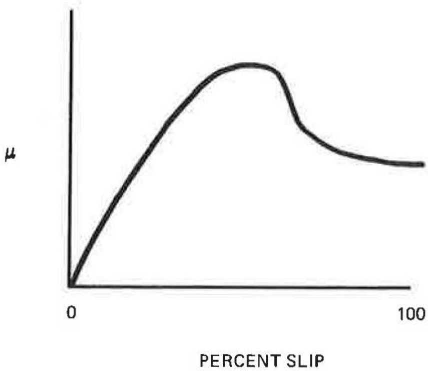

Figure 9. Typical curve of tire on wet pavement.

Figure 9 shows that the 100 percent slip value of μ is less than the maximum and corresponds to the locked-wheel case. If a driver applies the brakes such that the wheels continue to roll at the critical slip rate, he will generate a larger deceleration force than he will by locking the wheel. This will be true as long as the percentage of slip does not exceed that value of slip at the maximum μ. If it does, the wheels will lock and the μ will be reduced to that which exists at 100 percent slip. In some tires this value may

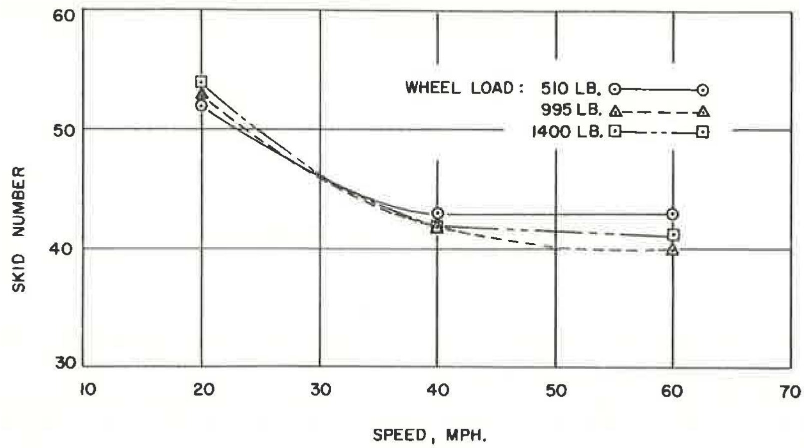

Figure 10. Skid number variation with wheel load and speed on wet concrete during locked-wheel test.

be up to 50 percent less than at the peak. The purpose of antilocking brakes is to prevent the percentage of wheel slip from exceeding the value at which maximum μ occurs. Such systems will ensure that vehicles will stop with maximum efficiency.

Suspension System—The suspension system on a car determines how the load shifts in cornering or stopping maneuvers. Although the lateral braking coefficient for each tire is affected by the load, the effects are not significant in the case of the longitudinal coefficient (16). Figure 10 shows the results of recent NBS work in which a skid trailer carried a wheel load of 500 to 1,400 lb. No significant differences in shape or slope were found.

In a maneuver that shifts 50 percent of its load off the rear axle onto the front axle, the potential braking coefficient as far as the vehicle is concerned may not be greatly changed. The danger is that the load distribution or shift may cause one of the rear wheels to lock at a relatively low brake force, which causes the driver to lose steering capability.

Braking Coefficient—It would be desirable to use the same trailers to evaluate the stopping capabilities of vehicles as well as pavement traction properties. Figure 11 shows the braking coefficient for an SAE tire as determined by both a trailer and a vehicle. The lack of agreement explains why it is unsafe to predict vehicle behavior from results obtained by using a trailer. Lack of correlation exists primarily because the trailer is maintained at a fixed speed, whereas the coefficient for the vehicle is calculated from the stopping distance in a locked-wheel skid with a given initial speed. In this case the vehicle's velocity varies from the velocity at wheel lock-up to velocity at rest. Air drag will influence the rate of deceleration.

Current work at NBS and elsewhere gives promise to the development of procedures that will give good correlation between trailer measurements and vehicle behavior.

Tire—Figure 12 (16) shows the slip curves for a belted-bias, a radial, and a bias-ply tire. The radial and bias-ply tires give very similar curves, whereas the belted-bias tire gives a higher value of friction coefficient at slip values of 10 percent and greater.

Figure 13 (6) shows a comparison of bald and treaded tires on two wet surfaces. On the smooth surface, the effect of tread grooves is more pronounced than on a skid-resistant surface.

Figure 14 (7) shows a comparison of the critical region for skidding on wet and dry surfaces. On a wet surface this region begins at a lower slip value than on a dry surface. On a dry surface, the braking coefficient curve drops very little after the maximum is passed. This means that near maximum efficiency in stopping is retained

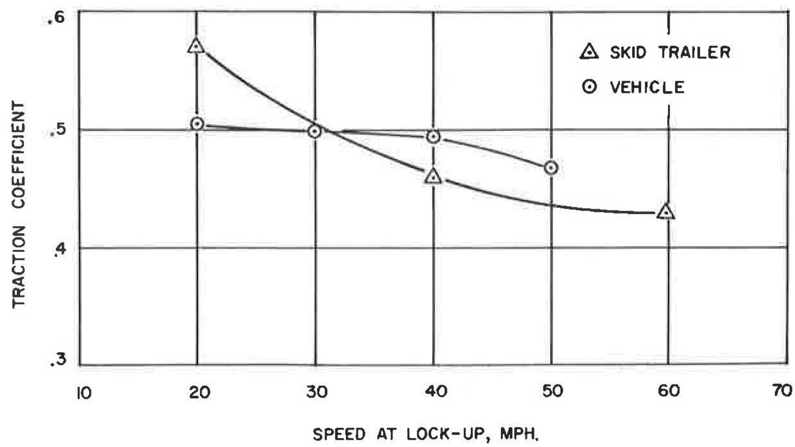

Figure 11. Comparison of traction coefficients by using SAE tires on wet concrete.

even if the wheels lock. With locked wheels, steering control will be lost on dry as well as on wet surfaces.

In the determination of skid number (SN) for a particular surface, care must be taken to ensure that the surface is free of contamination such as loose rocks, ice, or bacterial growth. Tests at NBS have shown a rapid increase in coefficient at the onset of testing on any site that has been free of traffic at least one night. Close inspection showed a fungus type of

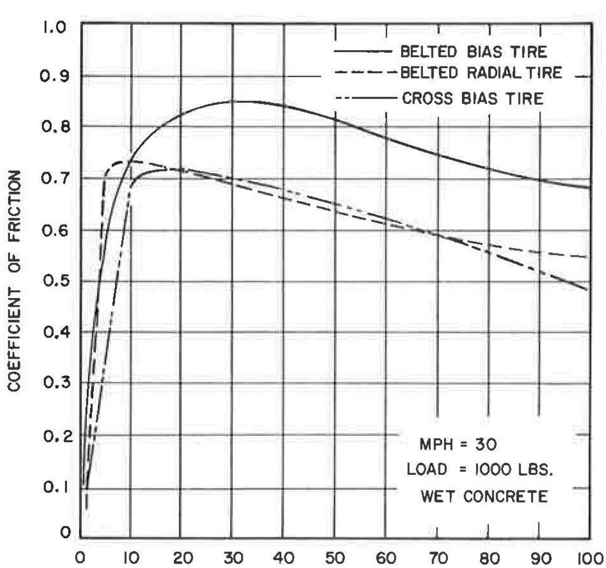

Figure 12. Effect of tire construction on slip curve.

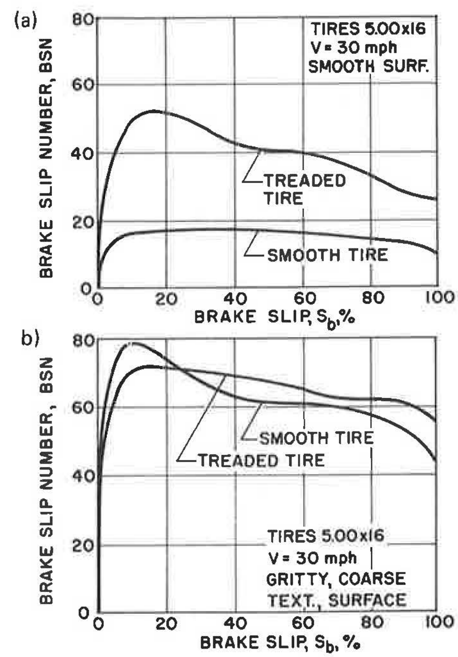

Figure 13. Typical brake slip resistance curves of slippery surfaces (top) and skid-resistant surfaces (bottom).

growth and dust on the pads, which was quickly burned or washed off.

Surface—The influence of surface type on available friction cannot be overemphasized. Kummer and Meyer (6) have defined five basic pavement types that exhibit extreme differences in the friction available at different speeds on wet surfaces. As shown in Figure 15, taken from their work, the very coarse, gritty surface (pavement 5) has the highest overall friction potential and the lowest sensitivity to speed—varying from a skid number of about 60 at 20 mph to 40 at 80 mph. Pavement 1, which has a very smooth surface, exhibits a low skid number at all speeds and an extreme sensitivity to sliding speed, approaching zero at a speed of 40 mph.

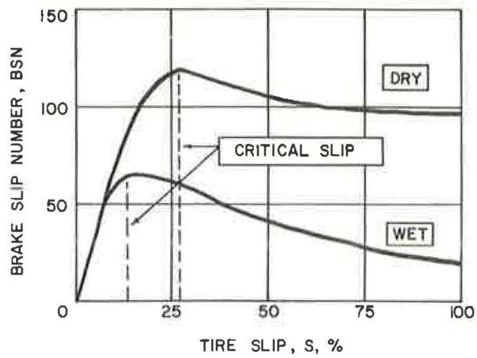

Figure 14. Relationship of friction and circumference slip.

There are some vary basic conflicts of interest among pavement designers, vehicle manufacturers, and tire manufacturers. A smooth, wet pavement will have the following effects on drivers, vehicles, and tires: (a) The hazard of a skid-initiated accident will be high; (b) the noise level will be low; (c) the rolling resistance will be low and fuel consumption will be low; and (d) tire wear will be low. A high friction pavement will have exactly the opposite influences: (a) The hazard of skid-initiated accidents will be decreased; (b) the noise level will increase; (c) the rolling resistance will be higher and fuel consumption will increase; and (d) tire wear will increase. Therefore, it would appear that a well-defined compromise may be necessary among safety, driver comfort, environmental effects, and vehicle-tire economy.

Probably the greatest cause of skid-initiated accidents is the spectacular difference between the friction available on pavements in dry and wet conditions. As shown in Figure 16 (6), available tire-pavement friction in the dry condition is very high and is influenced only slightly by vehicle speed. In contrast, the available friction deteriorates rapidly with the application of water and, depending on the type of surface, may be extremely sensitive to vehicle speed. On many surfaces the available friction is greatly reduced at the high speeds encountered on the Interstate System. Many engineers have stated that a film of dust and oil causes a very slick pavement condition when rainfall begins and triggers many skid-related accidents. Kummer and Meyer (6) have reported that they can find no documentation of this initial slipperiness. Gallaway (17)

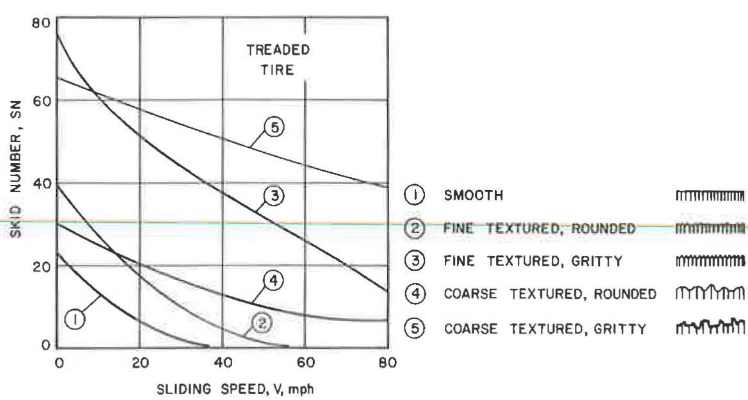

Figure 15. Available friction as a function of pavement texture.

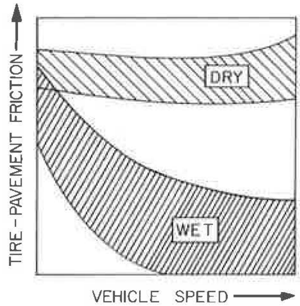

Figure 16. Sensitivity of friction to vehicle speed.

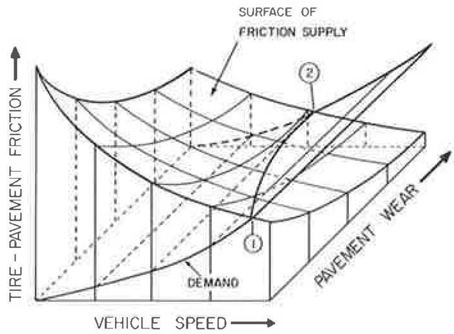

Figure 17. Comparison of friction supply and demand.

has stated that another contributing factor may be that drivers are unaware of the tremendous decrease in available friction that a little moisture can produce and therefore drive less conservatively when rain begins than they do after it has been falling for some time.

Figure 17 (6) shows a comparison of friction supply and demand as a function of vehicle speed and pavement wear. The height of the available friction surface decreases with vehicle speed and pavement wear. The construction of skid-resistant pavements involves techniques that are widely available. The construction of surfaces that maintain a high friction level during pavement life is a relatively new area. Considerable developmental work has been done by Gallaway (17), and a number of experimental highway test sections have been constructed. The results are very promising.

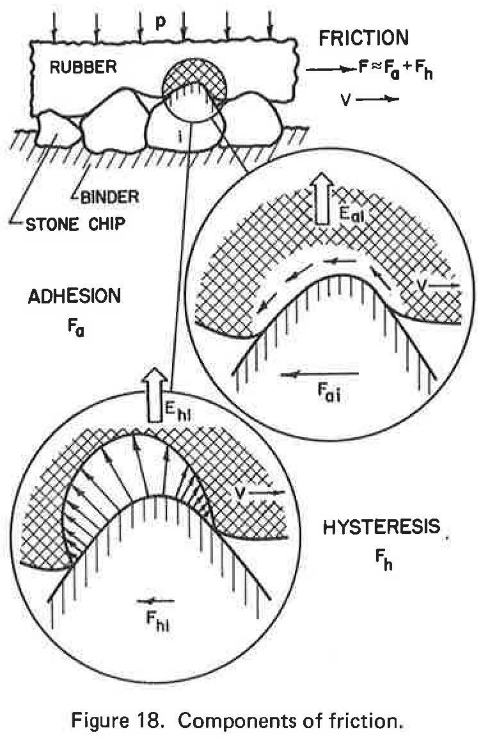

Figure 18. Components of friction.

An understanding of the basic mechanisms at work in producing friction forces between tires and pavements is essential to the discussion of building and maintaining high-friction surfaces. Almost all pavement surfaces are adequate when dry and clean. Therefore, consideration of the friction capabilities of dry, clean pavements is somewhat academic, and this discussion will be limited to wet pavements.

Figure 18 (6) shows the two principal factors, adhesion and hysteresis, that produce forces between a tire and a pavement. Adhesion is analogous to the force developed when a hand slides over the smooth surface of a desk. Adhesion is a microtexture interaction. If a paperweight is on the desk and the hand runs into it, the forces developed as the hand slides into, over, and out of contact with the paperweight are analogous to the forces causing causing energy absorption by the tire through hysteresis. Hysteresis is a macrotexture phenomenon that occurs when the tire is deformed by projections in the pavement, such as semi-exposed aggregate particles, and in the process absorbs energy.

When a pavement is covered with a layer of water, two events must occur in order for hysteresis and adhesion to exist:

1. The macrolayer of water must be expelled from the tire-pavement interface in order for the tire to be deformed by pavement macrotexture. The avenues for this water to escape are formed by the valleys of the pavement macrotexture and by the openings in the tire tread. As the speed of a vehicle increases, the time available to expel this macrolayer of water decreases. When there is no longer enough time to move this layer of water from underneath the tire, dynamic hydroplaning occurs and the tire is planing on a surface of water. This phenomenon is roughly analogous to water skiing.
2. The film, or microlayer of water, that remains on the pavement particles after the macrolayer has been expelled must be squeezed aside or penetrated by sharp aggregate projections so that adhesion between rubber and pavement materials can occur. As the speed of the vehicle increases, the time available to squeeze this film of water away from the aggregate particles decreases, and the available friction decreases due to loss of adhesion.

An excellent research program on tire-pavement interaction with very small layers of water (0.05 to 2.0 millimeters, or less than $\frac{1}{14}$ in.) has recently been reported by Gengenbach (14). Additional information on thin surface-water films was recently reported by Ludema (26). Work concerned with greater water thicknesses has been done by NASA and the Road Research Laboratory, but a significant program in this area is now being conducted at the Texas Transportation Institute (18). The difficulty that will be faced by the highway engineer in applying the information developed in these programs is that there is no such thing as constant water depths on highways. Every pavement section is a panorama of water conditions. The prediction of water layer buildup as a function of pavement slope, surface characteristics, and rainfall intensity is under investigation by Gallaway (19). The development of open-textured, self-draining pavements (20) may be an important breakthrough in the prevention of hydroplaning.

Gallaway has recently proposed several methods for prolonging or renewing skid resistance of the surface. He first published an article in 1967 defining aggregate produced for the refractoring industry and theorized that such materials would serve as nonpolishing aggregates for road surfacing purposes. Since the mid-1950s, researchers in the United States have experimented with manufactured aggregates and pavement materials. These manufactured aggregates have been produced by heating raw materials such as clays, shales, and slates in a rotary kiln to about 2,000 F. These aggregates have been referred to as lightweight or synthetic aggregates and are widely used in reinforced concrete construction. It was soon discovered that these aggregates have exceptional nonskid properties that are essentially independent of the volume of traffic on these surfaces. Figure 19 shows this type of nonpolishing aggregate and is taken from the work by Gallaway, who explains the formation of these high skid-resistant aggregates as follows (17):

> If one starts with the right raw material (clay, shale, or slate), and heats it to the pyroplastic state, gas in the form of very small bubbles will form within the heated particle. Bloating will occur. If the bloated particle is then cooled, the result is a stone or aggregate suitable for many uses such as lightweight masonry blocks, lightweight structural concrete and a vesicular aggregate highly suitable for producing non-skid pavements.

Other methods of producing prolonged skid resistance are: (a) the use of two aggregates that wear at distinctly different rates, (b) the use of natural material, such as sandstone, that renews surfaces by granulation, and (c) the dispersion of hard particles in a soft matrix. Some limestones contain discrete impurities in the form of hard particles dispersed throughout the softer limestone matrix. Some of the particles were described by James and are show in Figure 20 (21). The technology needed to greatly improve both initial and long-term skid resistance of pavements is rapidly becoming available.

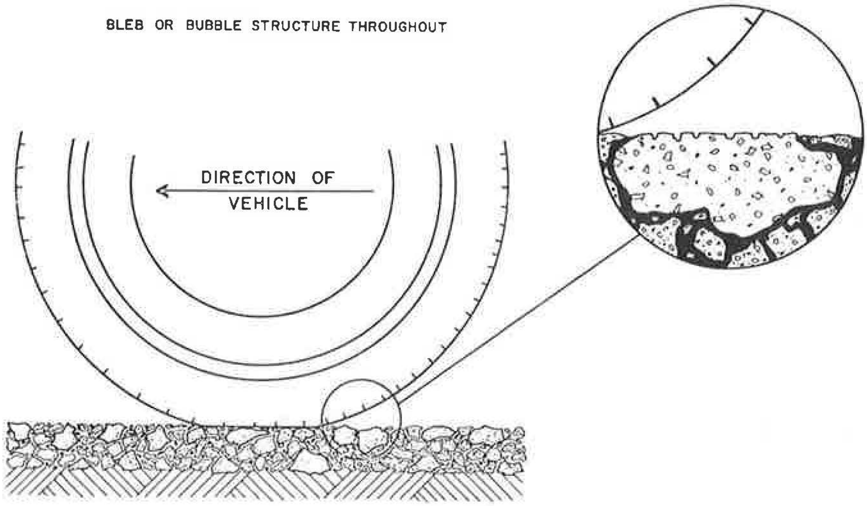

Figure 19. Lightweight synthetic aggregate.

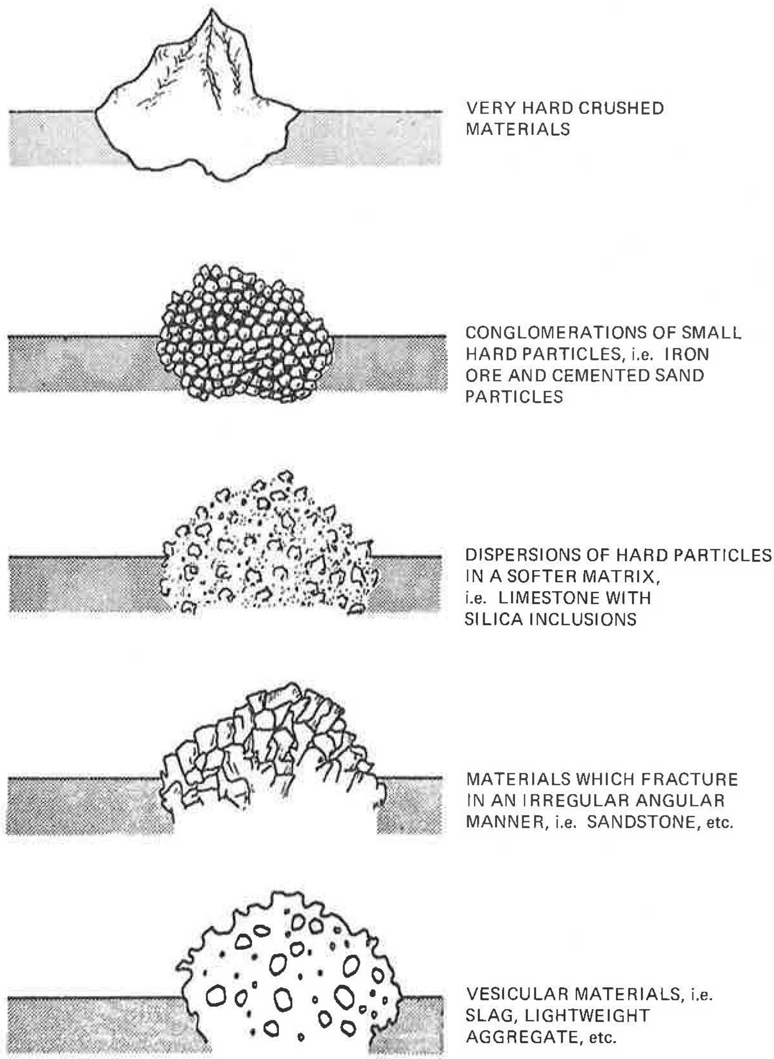

Figure 20. Theoretical types of polish-resistant roadstone.

SUMMARY

An accurate synthesis, or simulation, of the vehicle-pavement system that will reflect the interaction of the variables during vehicle maneuvers is critically needed. Important efforts in this area are being made by McHenry (22) at the Cornell Aeronautical Laboratory, by Dugoff (23) at the University of Michigan, and by Tiffany (24) at the Bendix Research Laboratory. Modifications to the McHenry program (CALSVA) are being made by Hirsch and Ross at the Texas Transportation Institute, in an effort to evaluate vehicle performance on highway curves. It should be emphasized, however, that these sophisticated computer simulations are based completely on empirically derived information. For example, variables such as vehicle spring constants, damping constants, moments of inertia, and tire-pavement force development under specified conditions must all be determined by tests before a vehicle can be simulated. Dugoff recently supported this by stating, "The principal difficulty in vehicle simulation is not in writing the equations of motion, but is in representing the subsystems of tires, brakes, and suspension." Significant progress is being made, and early results are very encouraging. It appears that a definite breakthrough in understanding the vehicle-pavement interaction will be forthcoming. Although a breakthrough of this sort will be most helpful to our overall analysis of the vehicle-pavement interaction, it should not be implied that such a breakthrough is necessary before meaningful action can be taken to alleviate problem areas. Considering the present state of knowledge, there is a great amount of technology that is ready to be applied.

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CORRECTIVE PROGRAMS FOR IMPROVING SKID RESISTANCE

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•THE major elements of a successful corrective program for improving skid resistance are (a) a responsible attitude toward providing skid-resistance roads, (b) a knowledge of the level of friction needed, (c) a reliable friction-measuring method, and (d) technical knowledge of the materials and methods used in providing corrective action.

RESPONSIBLE ATTITUDE

What is a responsible attitude toward providing skid-resistant roads? First, we must be convinced that slippery roads do in fact cause accidents. Frequently, we blame drivers for accidents that have been caused primarily by slick roads. If we cannot admit that roads are sometimes at fault, further research is futile.

Second, we must believe that the attempt to build high-skid-resistance roads is a good investment. Because it is our business to know about the dangers of driving on wet roads, we tend to minimize the hazards presented by such a driving situation. For example, we know that if we drive on wet pavements on well-worn tire treads, extreme caution should be exercized. To what extent is the public aware of this danger? To what extent do good tires lessen the danger?

Third, we must realize that certain materials tend to become slippery. Such materials must be used either cautiously or not at all in the construction of roads. This is not an impractical suggestion. For example, aggregates that should not be used in the surface courses of high-volume high-speed roads might be used in blends or alone on low-volume low-speed roads.

There are, of course, other items that help form what we call "responsible attitude," but suffice it to say that without the proper attitude our knowledge is of little value in developing a sound anti-skid program.

FRICTION NEEDS

Although agencies that have been working in the anti-skid field for some time are in general agreement that a skid number of 40 represents a road that is on the borderline of becoming slippery, frictional needs vary with different roads and under different circumstances. A list of the specific requirements of the various states has been prepared and indicates that some states do not have specific requirements for skid resistance but only adhere to certain guidelines (1).

At present the Franklin Institute is conducting a study to determine pavement friction coefficients required for the driving tasks. The specific objectives of this project are (a) to develop an approach for determining driver behavior under normal driving conditions and emergency or panic situations; (b) to conduct driver behavior studies and develop a procedure for determining frictional needs of traffic for any given condition or situation; and (c) to use the developed procedure to recommend minimum service values of skid resistance for general classes of highways and traffic conditions. Not only will the Franklin Institute give us recommended minimum service values for skid resistance of general classes of highways and traffic conditions, but it will also develop an approach that can be used by the individual states to determine what level of friction is needed on specific highways at specific locations. The project is scheduled to be completed in early 1972. This work is greatly needed and will be a giant step forward in the provision of skid-resistant roads.

Sponsored by Steering Committee for Workshop on Anti-Skid Program Management and presented at the workshop.

RELIABLE FRICTION-MEASURING METHOD

We are all familiar to some degree with friction-measuring methods such as the ASTM skid trailer, the stopping-distance, and the decelerometer. We need to improve those measuring tools and to increase our understanding of their output. At Pennsylvania State University, a study is in progress on locked-wheel pavement skid tester correlation and calibration techniques. The objective of this program is:

> . . . the development and verification of methods for improving the ability to measure the skid properties of pavement surfaces with skid testers constructed in general conformance with ASTM Method E 274-69. The research should be directed toward determining the reasons for the differences between results obtained with testers of essentially the same design and the resolution of these differences. Consideration should be given to such test and equipment variables as methods of wetting the test tire path, tester dynamics, and the lateral positions of the test tire path.

Individual agencies could make a worthwhile contribution if they researched the effects of water depth, temperature, and seasons on the friction values produced by the various test devices.

CORRECTIVE ACTION

There are six items that will be discussed in this section: superelevation, special spot improvement, resurfacing, pavement grooving, elimination of sharp curves, and "other" techniques.

Superelevation

In Virginia, the highway department alerts the Research Council when a location experiences skid accidents. These locations are checked with a skid-test trailer and are recommended for resurfacing if the skid number is 40 or below. On occasions we find that the skid numbers are rather high, but we suspect that some other condition, such as insufficient superelevation in a curve, contributes to skid accidents.

The AASHO curve design policy is based on the assumption that the curve is on level grade; that is, there is no vertical curve involved with a horizontal curve. At several locations where skidding accidents have occurred, the road had a skid number of more than 40 and the superelevation conformed to the AASHO design policy; however, these locations were on a right horizontal curve on a negative vertical grade. The grade was such that the elevation of the left front wheel was lower than that of the right rear wheel. Thus, a certain type of reverse superelevation resulted. This problem will be discussed in further detail later in the paper.

Special Spot Improvement

A good example of a special spot improvement occurred just recently. A plant mix that was placed during late summer became overly dense in spots, particularly in the wheelpaths of both the positive and negative grades on a hill. The weather was too cold to permit the resurfacing of these areas, and it did not seem to be the type of condition that could be corrected by a normal heater planing operation. A gang of teeth was dragged along the pavement under a heater planer, just in back of the flame. The results were excellent. The surface now resembles that of a grooved pavement, and it is felt that this location has certainly been made safe for the winter months.

Resurfacing

Historically, most of the roads that have been found to be slippery in Virginia have been made from polish-susceptible aggregates. For some time a policy has existed in Virginia that prohibits the use of mixes made of 100 percent polish-susceptible aggregates in the surface course of high traffic roads. The Virginia aggregates that have been found to be susceptible to polishing are limestones and dolomites, which are common throughout the valley area of the state. This area stretches from Winchester to

Bristol, a distance of 320 miles. The belt varies in width from 10 to 50 miles; however, it affects a belt of over 100 miles because quarry operators would much rather process the limestone than the harder, more skid-resistant aggregates. Consequently, 100 percent skid-resistant aggregates originally had to be imported into this area for the resurfacing of most primary highways.

However, to try to make use of local materials, the Council established a research project in which imported polish-resistant material was blended with the local polish-susceptible materials. The experiments confirmed the belief that a blend of these materials would provide a skid-resistant road if the coarse portion of the blend was composed of polish-resistant aggregates.

The blending of aggregates in the maintenance resurfacing program has met with great success and has been used extensively. For some time the polish-resistant aggregates used in the blends were natural materials, such as granites and sandstone, and most of them were imported from other parts of the state. More recently, lightweight aggregates have been used in the blends. The lightweight aggregates are expanded shales and are produced in several parts of the limestone area.

Three or four years ago the Council started experimenting with the sprinkle system. This is a technique in which highly skid-resistant aggregates are precoated with an asphaltic material and then sprinkled immediately behind the paver. Problems in setting up the machinery for this technique were anticipated, but a dump truck equipped with a chemical spreader was backed up to the paver without causing damage. In the sprinkle system, 100 percent limestone is used in the hot plant mix and some 3 to 5 lb/yd^2 of the skid-resistant aggregate is sprinkled on top of it. The skid-resistant aggregates that have been used are both natural and lightweight. This method also has met with great success.

For many years a thin sand overlay has been used to deslick pavements in Virginia (2). Many other states use thin sand overlays and all seem to have encountered one problem: Occasionally on high-speed roads with heavy traffic volumes, the mix undergoes scaling. It is believed that scaling is a result of stripping caused by hydrodynamic pressures.

Another type of resurfacing that seems to hold great promise is an open type of mix. Engineers in Texas have been working in this field for several years; and it would seem that, if strength and durability could be incorporated into this type of mix, it would contribute greatly to the building of skid-resistant pavement because of its excellent water draining properties.

Another resurfacing technique is the application type of surface treatment. This technique has been used very little for deslicking pavements in Virginia because high traffic volume roads are Virginia's greatest problem with skidding accidents. Also, Virginia has not been able to develop surface treatments on high traffic volume roads to the point where they are durable enough to be economical.

Pavement Grooving

In areas where wet-weather accidents have become prevalent, many state highway departments have been grooving their pavements in an effort to increase skid resistance and thus reduce accidents. Where accident data are available, it can be shown that the numbers of accidents have been reduced by 30 to 60 percent.

Pavement grooving is the process of making shallow cuts of a uniform depth, width, and shape in the surface of a pavement. Grooving differs from texturing in that the latter is a process of lightly sawing or scouring a pavement to expose a new surface. The terminology associated with pavement grooving includes the terms pitch, width, and depth. The pitch is the distance between the grooves, the width is the groove opening, and the depth is measured from the pavement surface to the bottom of the groove. Pavement grooving is usually accomplished by automated equipment that uses diamond-studded blades gang-mounted on a single shaft and that cuts a path about 2 ft wide. The machines are capable of grooving at a rate of 5 to 30 ft/min depending on the hardness of the pavement aggregate and the number and size of grooves.

Pavements can be grooved either transversely or longitudinally, but more often they are grooved longitudinally. Grooves placed longitudinally with the roadway have proved to be the most effective in increasing the directional control of a vehicle. The automobile tires apparently penetrate into the grooves and form a mechanical interlock that helps hold the vehicle in alignment with the roadway. Where heavy films of water are encountered, the grooves tend to channel the excess water away and thus reduce or eliminate the potential for hydroplaning.

The grooving process is practical on roadway surfaces where the skid resistance is slightly below an acceptable level; it is not generally satisfactory for very slippery pavements. Pavement grooving may be beneficial in several typical situations: in areas where the geometric alignment of the roadway contributes to a rash of wet-weather accidents; in areas where heavy rainfall causes excessive water depths; on pavement locations frequently subjected to strong crosswind currents; and at locations such as sharp exit ramps where there is a quick transition from high speed to a sharply reduced speed coupled with turning.

Although agencies have reported that pavement grooving reduces accidents, the same agencies report that skid resistance may not be significantly changed. This is probably because most skid trailers measure only the braking coefficient of friction and do not indicate side slip or cornering traction. Tests conducted at Wallops Island by the National Aeronautics and Space Administration indicated that, although longitudinal grooving increased the braking coefficient of friction by only a small amount, the cornering traction around a 500-ft radius curve was 3 to 4 times more than that obtained on an ungrooved pavement.

Several factors affect the performance and durability of the grooving. The groove spacing is important; the closer the grooves are, the more apt the pavement surface is to spall. There seems to be no problem, however, when the grooves are spaced on ¾-in. centers. If an aggregate is soft, the grooves will not stand up under the effect of heavy traffic, particularly under chains and studded tires. Although grooving has been associated chiefly with concrete pavements, it is entirely possible to groove bituminous surfaces providing they are old pavements with a high aggregate content. Soft aggregates in both bituminous or portland cement concrete surfaces may contribute to a limited effective service life of only 6 months under moderate to heavy traffic.

Frequently, concern has been expressed over the effect of water freezing in these pavement grooves, but both laboratory and field observations indicate that this condition does not produce spalled concrete in the shallow grooves. Also, traffic has a natural tendency to keep the grooves clean. Some complaints have been encountered regarding the effect of grooving on vehicle steering, but this problem does not appear to exist when the grooves are kept in perfect alignment. A more severe problem associated with pavement grooving is the resulting slurry, which may run across an adjacent pavement lane, or the resulting dust when the slurry dries. These hazards can be eliminated by specifying a slurry pickup device on the cutter, which pumps all debris to the shoulder area.

Pavement grooving has apparently been accomplished in a good many states with notable success. It can be expected that average costs for this service will run between \$0.95 to \$1.30/yd^3 depending on the area and the availability of contractors. Several specialists are available for this service, and some local contractors can also perform this work. The grooved pavement should perform satisfactorily for several years before retreatment is required.

Elimination of Sharp Curves

It is common knowledge that hills, curves, and heavy volumes of traffic contribute to high accident rates. These specific troublesome areas are generally pinpointed from an analysis of traffic data accumulated by accident review teams, traffic analysts, or state police. In many instances, it is perfectly obvious that a sharp curve is a problem area, and analysis is not necessary.

The Interstate and primary highway systems constructed in recent years are generally free from such deficiencies because of design criteria. However, older and secondary road systems present a problem. The highway designer can contribute much to the solution of this problem by providing a highway of uniform design; that is, a driver experiencing uniformly moderate curves should not be expected to negotiate a suddenly encountered sharp curve without trouble. The radius of curvature should not suddenly change within a curve, particularly on a superelevation.

Some curvature problems can be corrected by local anti-skid treatments; but if the problem is severe, realignment may be the only solution. If the area is hazardous, realignment should definitely be programmed. There is no way to evaluate the potential cost of such an undertaking on a large scale because many factors would have to be considered and each location would have to be evaluated separately for the type and extent of redesign and reconstruction required.

Other Techniques for Providing Skid Resistance

Several agencies have reported limited success with the use of chemical compounds that etch the pavement surface. These are generally concentrated solutions of hydrochloric or hydrofluoric acids that are diluted with water in the field and applied to the pavement surface with simple spray equipment. After a period of initial reaction, the solution is washed from the roadway surface. No pretreatment of the pavement such as washing or brushing is required. As a result of the chemical reaction, the pavement surface takes on a sandpaper texture and is cleansed of oil deposits and other dirt. The cost is relatively low, approximately 4 to 5 cents/yd^2 for materials alone. The results are not very durable, however, and an improvement in skid resistance is generally limited to a 6-month period; therefore, the treatment has to be applied once or twice yearly.

The use of this material can be dangerous. The concentrated acid is very strong, and even outdoors it must be handled with extreme caution. Heavy, protective clothing must be worn by the workmen, and masks must be worn to prevent the inhalation of dangerous fumes. This process should be undertaken only with extreme caution.

Other materials used in the restoration of pavement surfaces are synthetic aggregates and undeveloped highly skid-resistant natural aggregates. Some research has been conducted on the use of synthetic aggregates, and the resultant skid resistance has been good. However, these synthetic materials are usually made from high silica raw materials and this material, when encountered in natural aggregates, also produces good skid resistance. Therefore, it appears that equally good skid resistance can be attained from both high silica, synthetic aggregates and natural aggregates. At present, the cost of the synthetic aggregates is quite high. Highway departments should be urged to encourage the development of untapped aggregate sources that have potentially good skid-resistance properties. These sources have not been developed, in many instances, because of the necessary costs in opening a source and in crushing and processing a harder than average material. As our natural aggregate supply dwindles, we must encourage the development and production of new sources and generally expect that the costs are going to be somewhat higher than previously encountered. The improvement will be well worth the cost.

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ANTI-SKID MEASUREMENT

R. S. Millard, Road Research Laboratory, Great Britain

•THE basic needs of the anti-skid program are to define suitable criteria for the skid resistance of road surfaces and to provide the engineer with a convenient method of measurement.

In Great Britain the sideway-force coefficient (SFC) is used for routine measurement of skid resistance because the measurement of most significance is the one at which skidding begins and because reasonably reproducible results can be obtained.

The sideway-force coefficient routine investigation machine (SCRIM) has been developed and consists essentially of a standard four-wheeled vehicle chassis carrying a fifth test wheel set of 20 deg to the direction of traffic, a water tank, and a data logger with printed output. The machine records the SFC at 10- or 20-meter intervals, can test at speeds up to 60 mph in the traffic stream, and can cover up to 80 miles of road in one day.

Criteria have been defined to provide the highest resistance to skidding where the risk of skidding accidents is greatest. At the most dangerous sites, e.g., on approaches to busy junctions, the criterion is 0.55 SFC at 30 mph. At average sites, e.g., on motorways or on other high-speed roads, criteria are 0.5 SFC at 30 mph and 0.45 SFC at 50 mph.

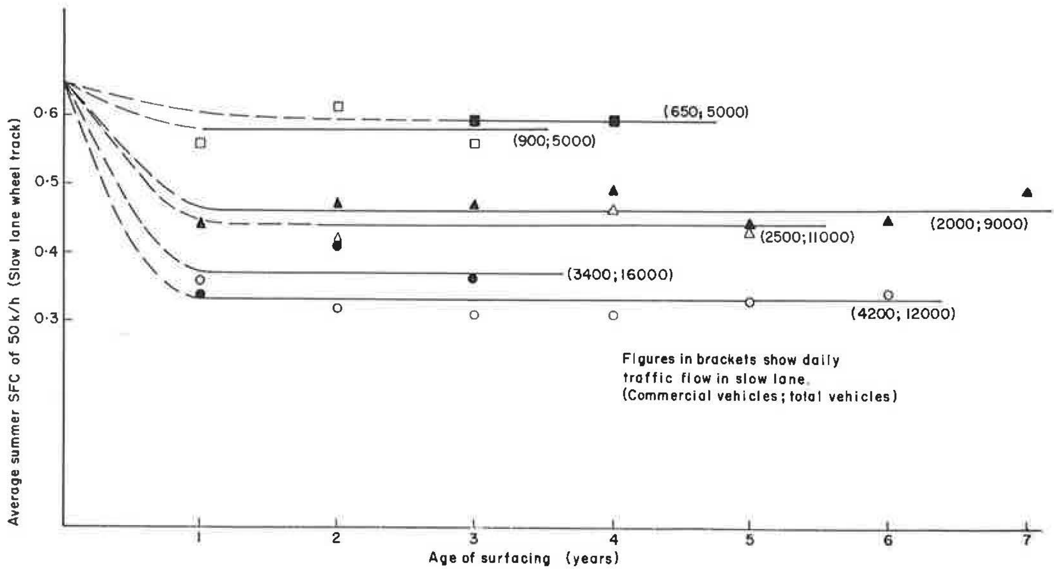

Figure 1. Effect of traffic on mean summer sideway-force coefficient.

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It has recently become evident that, even when the best natural aggregates are used, a high resistance to skidding cannot be maintained under heavy commercial traffic. Figure 1 shows that with a given road surfacing, in this case a rolled asphalt with pre-coated chippings of high resistance to polishing (the typical surfacing used on heavily trafficked roads in Great Britain), the resistance to skidding is dependent entirely on traffic intensity.

Thus the present criteria must be regarded as targets, and they will be revised as more is learned about the cost of maintaining skid resistance at different levels and the associated savings in accidents. There is also a need to develop synthetic aggregates that are highly resistant to polishing.

SPRAY GRIP ANTI-SKID SURFACING MATERIALS

Alex J. MacKenzie, Contract Services, Prismo Universal Corporation, Rockville, Maryland

•THE use of spray grip anti-skid surfacing materials is a new form of surface treatment that has proved extremely successful on accident-prone spot locations, such as intersections, where there has been a high frequency of wet-road skidding accidents. In London, where 41 sites were treated, rear-end collisions were reduced by 73 percent during a 2-year period.

The key factors contributing to the performance of spray grip materials are binder, aggregate, and application process.

The binder is a bitumen or asphalt extended epoxy that has a tensile strength averaging 2,000 psi. This provides the adhesive qualities necessary to hold the aggregate in place on the road and in the same attitude; that is, it prevents the particles from rotating and offering a smooth face to the tire. A necessary feature is the percentage of elongation that the material can offer.

Volume of Part A Resin to Part B (percent)	Tensile Strength (psi)	Elongation (percent)
47½	1,500	70
50	2,000	50
52½	2,500	30

The tensile strength can be varied depending on the ratio of parts A and B of the two-part resin. The optimum reading with a 50:50 ratio, which gives a tensile strength of 2,000 psi, is 50 percent elongation. These results are obtained when testing in accordance with ASTM D 638 at ¼ in./min to 250 lb. The elongation factor allows the necessary flexibility in the aggregate particle, which aids durability and eliminates shock removal.

The aggregate is RASC grade calcined bauxite mined in British Guyana, calcined at 1,600 C. This is the only material we have found to be satisfactory in practice. There is no laboratory test for quick acceptance of any other material. The gradation for the bauxite material is as follows:

U. S. Standard Sieve Size		Percentage	
Passing	Retained	Minimum	Maximum
—	6	0	3
6	7	5	15
7	16	80	90
16	30	5	15
30	50	0	5

This is the minimum size needed to give satisfactory texture depth over a reasonable life.

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The application equipment was designed and built by Prismo engineers and provides the necessary controlled flow conditions to ensure the correct ratio of part A to part B. One of the essential prerequisites is that the binder be preheated to accelerate the curing time. Cure time is dependent on road temperatures and varies from 2 to 5 hours. The road must be kept free from traffic until the curing is complete. Excess bauxite is swept up after curing to ensure that the site is skid resistant immediately and to enable all surplus to be reused because the aggregate is a very expensive part of the total cost.

Some sites have been treated in the United States including the toll plaza northbound on the Delaware Memorial Bridge, and initial reaction from engineers has been most enthusiastic. The process was first applied in the United Kingdom in 1967, and these first applications are still performing well.

Skid test readings of more than 70 after 1 year have been obtained on the British portable tester; initial readings on the tester at a site in North Carolina were 97. The main advantage of the process is that high readings are obtained throughout the life of the installation.

If the sites are carefully selected and the area to be treated is kept to the effective minimum (i.e., 200 ft up to the stop line on approaches to intersections with a 30-mph speed limit should receive a surface dressing), the reduction in accidents will more than justify the cost of spray grip materials.

GROOVING PATTERN STUDIES IN CALIFORNIA

George B. Sherman, California Division of Highways

•GROOVES can be cut into pavement in a longitudinal (parallel to the centerline), transverse, or skewed direction. All grooving (except for a few short experimental sections) to date on California highways has been performed in a longitudinal direction. We are of the opinion that this leads to increased lateral stability and tends to guide the vehicle through a critical curve area. The entire lane width is grooved except for about 1 ft adjacent to each lane line or edge of pavement. This is done to preserve the lane line or markers and also to permit the use of vacuum devices that remove water and cutting residue concurrently with the grooving operation. This practice greatly reduces the hazard to traffic during the grooving operation.

Several patterns have been used in serration work. Width and depth of grooves have varied from $\frac{1}{4}$ in. to slightly less than $\frac{1}{8}$ in. (0.095 in.), and spacing of the cuts has varied from $\frac{3}{8}$ to 1 in. center to center. This was done in order to determine the increase in friction factor, wear resistance, and possible vehicle handling problem of various patterns. In all cases the grooves were made in a longitudinal direction.

The various grooving patterns that have been explored and comments on their effectiveness follow.

The closest spaced pattern ever used in California was $\frac{1}{8}$ in. by $\frac{1}{8}$ in. at $\frac{3}{8}$ in. centers. First used in 1960, this pattern was effective in raising the coefficient of friction; however, it caused spalling and was quite expensive to put down. It was used only three times.

Another pattern, $\frac{1}{8}$ in. by $\frac{1}{8}$ in. at $\frac{1}{2}$ in. centers, has been used effectively since 1963 in increasing the coefficient of friction. It will spall if cut too deep; however, if a 0.095-in. blade is used for this pattern there is less tendency to spall. This pattern, cut with the 0.095-in. blade, was used in 1970 on a road where the coefficient of friction was very low. It is an excellent pattern for motorcycle ridability.

The standard California pattern is $\frac{1}{8}$ in. by $\frac{1}{8}$ in. at $\frac{3}{4}$ in. centers, used first in 1966, or the thinner blade of 0.095 in., used first in 1968 to improve motorcycle ridability. This pattern has done an excellent job.

The use of two patterns, the first $\frac{1}{8}$ in. by $\frac{1}{8}$ in. at 1 in. centers and the second $\frac{1}{4}$ in. at 1 in. centers, has been discontinued. The former is a good pattern for motor-

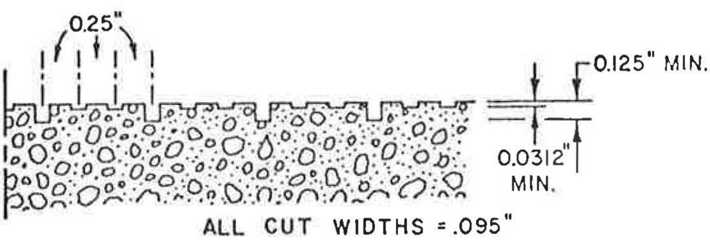

Figure 1. Style A grooving pattern (first used in 1970).

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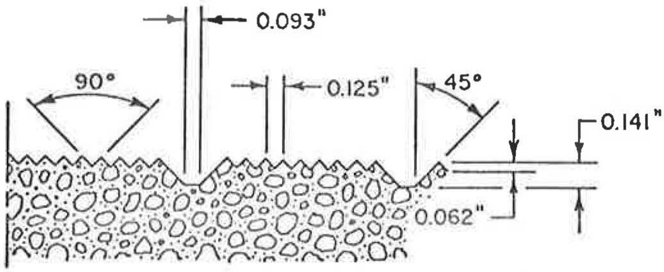

Figure 2. Style 15 grooving pattern (first used in 1968).

cycle ridability but does not raise the coefficient of friction appreciably. The latter is a standard airfield pattern but results in extremely poor ridability for both automobiles and motorcycles.

Style A grooving pattern (Fig. 1) was tried in areas having a low coefficient of friction. Because of the roughness of pavement surfaces, however, it was practically impossible to put this pattern down and to comply with depth specifications.

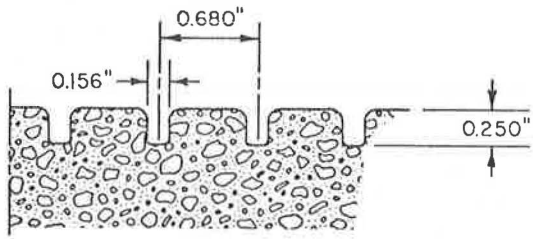

Figure 3. Style 9 grooving pattern (first used in 1967).

Style 15 (Fig. 2) is termed a "molded-head" pattern. Although Style 15 does an excellent job of increasing the coefficient of friction, it is a difficult pattern to apply and results in poor motorcycle ridability. This pattern is patented by Christensen Diamond Services Company.

Style 9 (Fig. 3) is also a "molded-head" pattern. Like Style 15, this pattern increases the coefficient of friction but results in poor motorcycle ridability. It has no advantages over a pattern cut with standard blades. This pattern is also patented by Christensen Diamond Services Company.

STATUS OF SKID MANAGEMENT PROGRAM IN TEXAS

John F. Nixon, Texas Highway Department

•THE Texas Highway Department has recommended consideration of legislation to establish wet-weather speed limits and requirements for a minimum tread depth. In addition, television spots and movies are being prepared to inform the public of the hazards of wet-weather driving.

Highway improvement procedures being made in Texas include the following:

1. A minimum coefficient for use as a guideline for maintenance has been established, but because of liability implications it is not a construction requirement;
2. A method of skid-trailer measurement has been initiated;
3. A statewide inventory system for measuring skid coefficients was started 3 years ago;
4. Specifications that require a minimum polish value of the coarse aggregate as determined by the British accelerated polish procedure and that take into consideration the differential weight of synthetic aggregates by measuring ACP by volume in place are being used experimentally;
5. Although a construction requirement does not exist, more macrotexture is being used in the construction of pavements;
6. Procedures such as grooving, geometric design, and accident-site investigation have been initiated;
7. Stopping sight distance requirements have been increased;
8. Minimum cross-slope requirements may be increased to aid in decreasing water depth;
9. Research is under way regarding hydroplaning, vehicular operation on curves, polishing of asphaltic mixtures, concrete pavement finishing, systematic evaluation of factors affecting vehicle skidding, and cross slope and texture with regard to water depth in various intensity rainfalls;
10. The feasibility of using cost-effectiveness procedures in selecting sites to perform remedial measures directed toward reducing accidents is being studied; and
11. Methods are being studied for improving the inventory system whereby skid resistance, traffic, and accident performance records may be combined, maintained, and analyzed.

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INFLUENCE OF VEHICLE AND PAVEMENT FACTORS ON WET-PAVEMENT ACCIDENTS

Kenneth D. Hankins, Richard B. Morgan, Bashar Ashkar, and Paul R. Tutt,
Texas Highway Department

Five variables believed to be closely associated with the friction available at the tire-pavement interface were analyzed by studying 501 wet-weather vehicular accidents. Tire pressures and tread depths were obtained from the accident vehicles, vehicular speed from the investigating officer's report, and friction and macrotexture from the pavement surface at the accident site. It was concluded that the lack of pavement texture, low pavement friction, high vehicle speed, worn tires, and large vehicle tire pressures all contribute to accidents occurring on wet pavement. The accidents were also categorized into several types, and it was found that the variables are even more significant for certain accident types. Studies should be directed toward cornering friction because some 40 percent of the accidents involved a turning maneuver.

•THE research reported in this paper attempted to determine the effect of certain vehicle and pavement factors on traffic accidents occurring during wet weather. The detailed roadway and wet-weather accident data investigated and analyzed were collected by the Texas Department of Public Safety and the Texas Highway Department in a joint effort that represents the desire of both agencies to reduce the number and severity of accidents that occur on Texas highways.

This report concerns only a portion of the data collected. Five variables were arbitrarily selected for reporting because they were thought to be closely associated with the tire-pavement interaction related to vehicular skidding on wet pavement. The variables analyzed were as follows:

Number	Description	Code
1	Vehicular speeds at the time of or immediately prior to the accident	SP
2	Tread depths of the tires of the accident vehicles	TD
3	Tire pressures of the accident vehicles	PR
4	Pavement friction (skid number) at the accident site	FR
5	Pavement macrotexture at the accident site	TX

The object of the analysis was to determine the degree of influence of each variable on wet-weather accidents.

DATA COLLECTION

The data were collected from 501 wet-weather accidents that occurred from May 1968 through September 1969 in an area that consisted of 10 central Texas counties and covered portions of two highway districts (Fig. 1). The area contained approximately 2,460 miles of the state's highway system, on which the average daily vehicle travel was 3,086,099 miles (1).

Sponsored by Steering Committee for Workshop on Anti-Skid Program Management and presented at the workshop.

The study area was primarily rural in character. Urban areas with populations of more than 5,000 were not included to simplify data collection; accident investigation and reporting for these areas are the responsibility of the municipalities involved rather than of the Texas Department of Public Safety.

Accident Data

The data were collected by the Department of Public Safety (DPS) and the Texas Highway Department (THD). The DPS, in addition to making the usual accident investigations and gathering documentation for the standard report, collected information for a second special report that contained the tire pressures and tread depths of the accident vehicles.

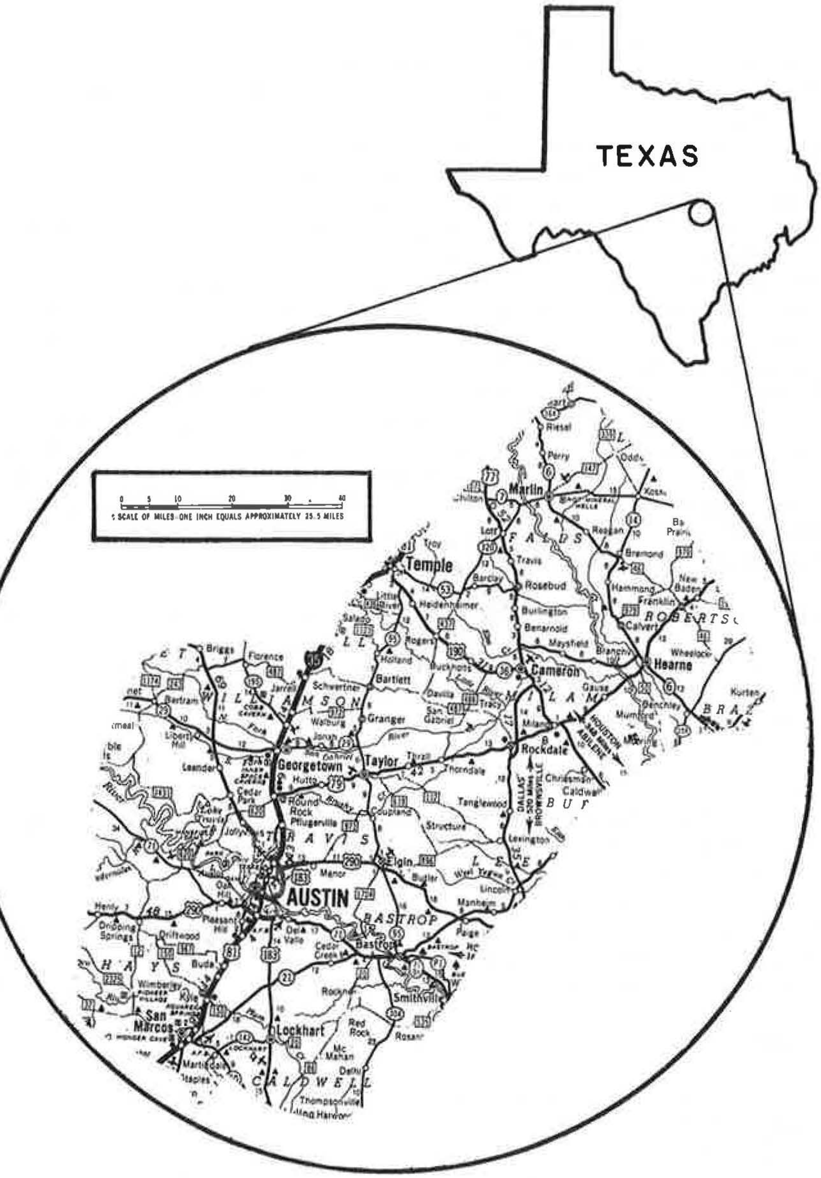

Figure 1. Location of study area.

The pressure for each tire was measured with a commercially produced tire-pressure gage with 2-psi divisions. (It was calibrated before use.) In several instances a tire was so damaged that it was completely deflated, and often a tire appeared to be partially deflated as a result of the accident, especially when a broadside skid had occurred.

The tread depth of each tire on a vehicle involved in a wet-weather accident was measured to the nearest $\frac{1}{32}$ in. with commercially produced tread-depth gages. The minimum and maximum tread depths were reported for each tire. One measurement was made near the edge of the tire and the other near the center of the tire.

The speed of the vehicle immediately prior to the accident was obtained from the standard investigating officer's report. DPS officials indicated that reported speeds could vary from actual speeds and could be biased by the individual officer. They are believed to vary ± 10 mph and generally to be lower than the actual values for speeds above 50 mph.

As soon as possible after an accident, THD personnel investigated pavement and roadway conditions at the accident site, which had been conspicuously marked by the DPS investigating officer. Skid resistance was measured with a skid test trailer and texture with a modified Southwest Research Institute (SWRI) texturemeter.

In this project, attempts were made to treat surface macrotexture and the skid number as separate entities. This was accomplished by measuring both variables with the separate instrumentation previously mentioned.

Usually, a skid test was made at the marked location, and three more tests were made over approximately $\frac{1}{2}$ mile in both directions from the accident site (a total of 7 skid tests). On nondivided highways, tests were normally made on lanes in both directions of travel. On divided highways, tests were usually made on all lanes in one direction of travel, but lanes in both directions were tested if vehicles involved in the accident were traveling in opposite directions.

Friction measurements were obtained at the accident site and $\frac{1}{2}$ mile in either direction from the site because the initial postulation had been that a vehicle and driver could be affected by a sudden change in pavement friction; that is, if a vehicle traveling on a pavement with good friction characteristics suddenly maneuvered onto a section of pavement with poor friction, there could be problems in controlling the vehicle.

Skid-resistance measurements were made at 50 mph. Occasionally the accident location, such as a T-intersection, made skid testing unfeasible.

Texture measurements were made at two locations in an area in which a vehicle lost control, and the average of the two measurements was reported. The readings were obtained with a modified version of a profilograph instrument developed at SWRI (2). The texture equipment used was described in a previous report and is actually a mechanical instrument that records the cumulative of the asperities of the texture and scribes a magnified profile of the texture.

Comparison Sample Data

In order to have information with which to compare the accident data and to determine how accident conditions differed from normal driving conditions, special sample data were gathered for each of the five items under study.

Prior to the research reported here, very little information had been gathered on wet-weather driving speeds in Texas. The information contained in this report was obtained by the THD Design Division, Geometric Design Section, which monitored speeds on rural highways in the study area zoned at 70 mph, during periods of rainfall, with radar speed indicators. There was some doubt that the radar would read correctly in rainfall, but a test car was driven through the radar site during the rain and correct readings were obtained.

Sample tread depths and tire pressures from 250 parked vehicles were obtained in two large cities in the study area, Austin and Bryan, and in two smaller cities, Rockdale and Smithville.

A sample of the friction on the highways in the area was collected from routine friction tests performed during the period of this study. These routine tests were performed at 40 mph rather than at the 50-mph test speed maintained at the accident sites. Experience in performing skid tests over the same test section at different speeds indicates friction differentials on the order of 0.02, or two skid numbers generally occur between 40 and 50 mph. Because of this small friction differential, no attempt was made to correct the comparison sample or the accident sample to a constant velocity.

The texture sample for the area highways was determined from the type of pavement and the type of coarse aggregate used on the surface. The pavement and aggregate types were determined for every highway in the study area. The length of each highway segment containing a specific pavement material type was recorded with the average daily traffic count for each segment, and a daily vehicle-miles of travel was calculated for each type. A texture value for each pavement material type was obtained from another publication (3). Substituting texture values for pavement material types gave the daily vehicle-miles of travel for each texture group.

ANALYSIS

The relationship of the accident data and the area sample, representing the normal driving conditions, is shown in Figures 2 through 12.

Figure 2 shows a comparison of the speeds of the accident vehicles and the area sample vehicles. The accident vehicle generally traveled at a slower speed. For example, only 2 percent of the vehicles in the area traveled at speeds of 30 mph or less, whereas approximately 8 percent of the accidents occurred at speeds of 30 mph or less. It must be admitted that this condition was not expected; however, it was noted that many of the accidents reported at low speeds were rear-end collisions in which one vehicle had slowed for a turning maneuver.

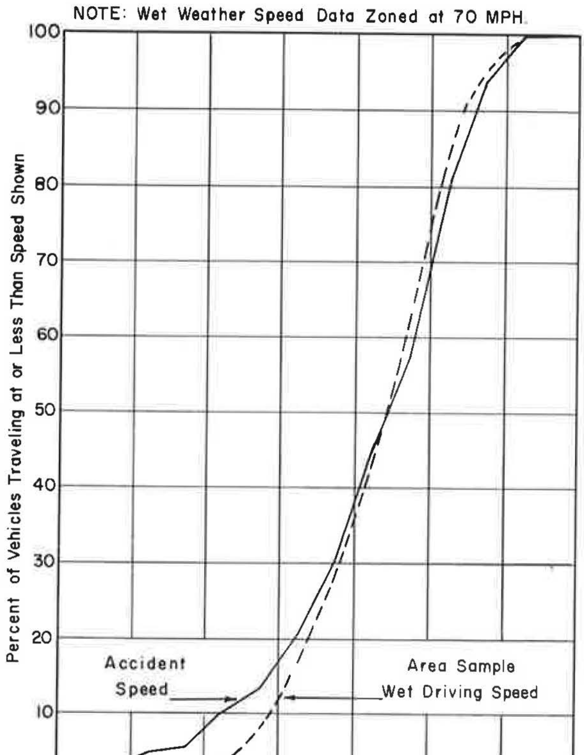

Figure 2. Comparison of vehicular speeds.

Figures 3 through 6 show that accident vehicles generally had less tire tread depth compared to the average vehicle in the study area. This is particularly true for the rear tires; about 10 percent of the rear tires of all accident vehicles were completely smooth.

The same general trends are shown in Figures 7 through 10 with the exception that there is a considerable increase in the number of accidents involving vehicles containing tire pressures of 28 to 29 psi.

As stated previously, the skid number was measured with a trailer at the accident site and in the ½-mile area in both directions from the site. Dean (4), in a previous report for this project, has shown that, statistically, there is no significant difference between the skid number of the average site (35) and that of the average of a statewide sample consisting of more than 2,000 sections (39). It has since been found that the average friction for a sample of pavements in the study area is also 39, with the variance in the study area similar to the statewide variance. The average for the 1-mile vicinity

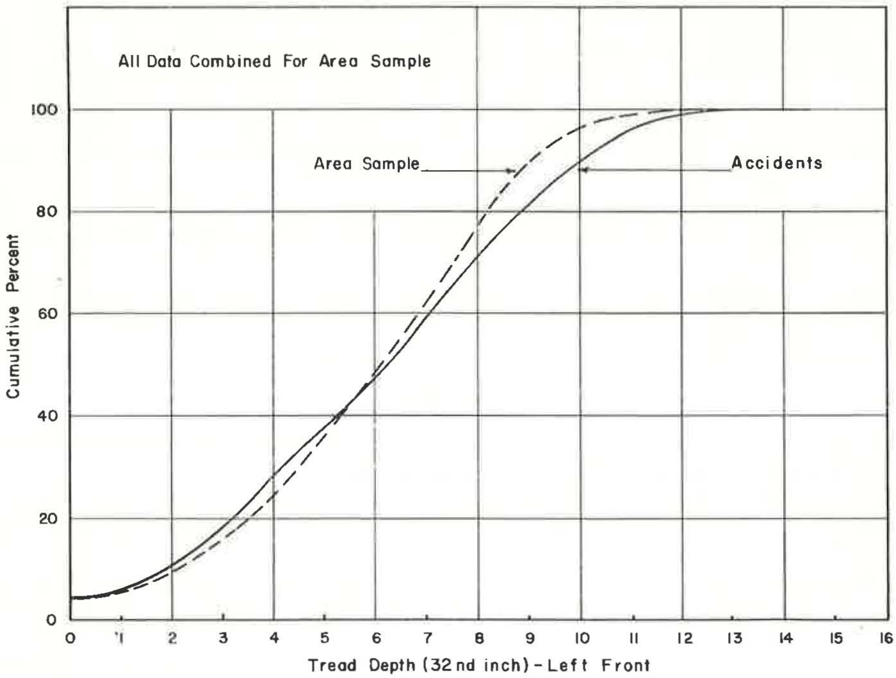

Figure 3. Comparison of left front tire tread depths.

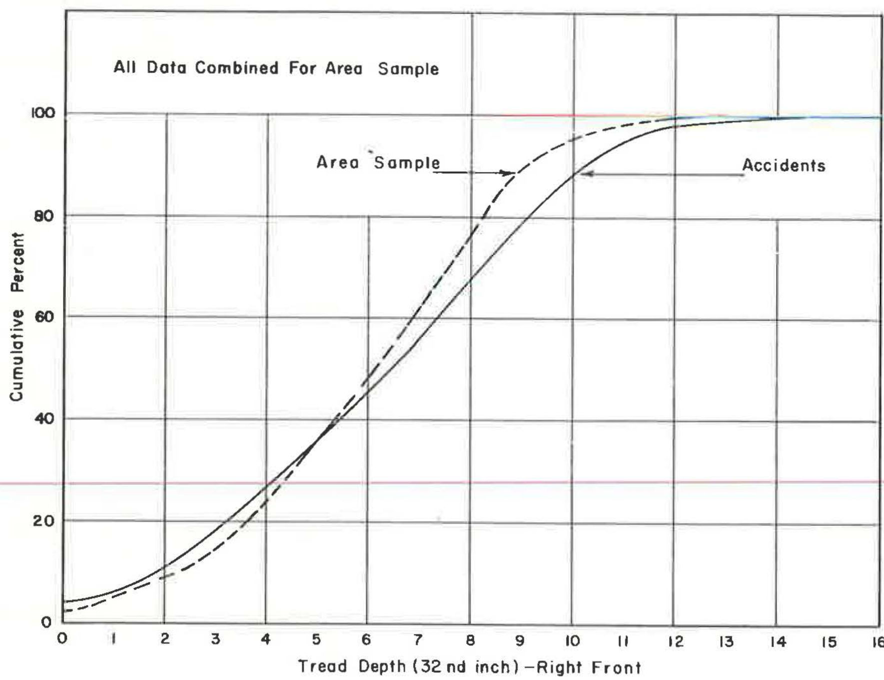

Figure 4. Comparison of right front tire tread depths.

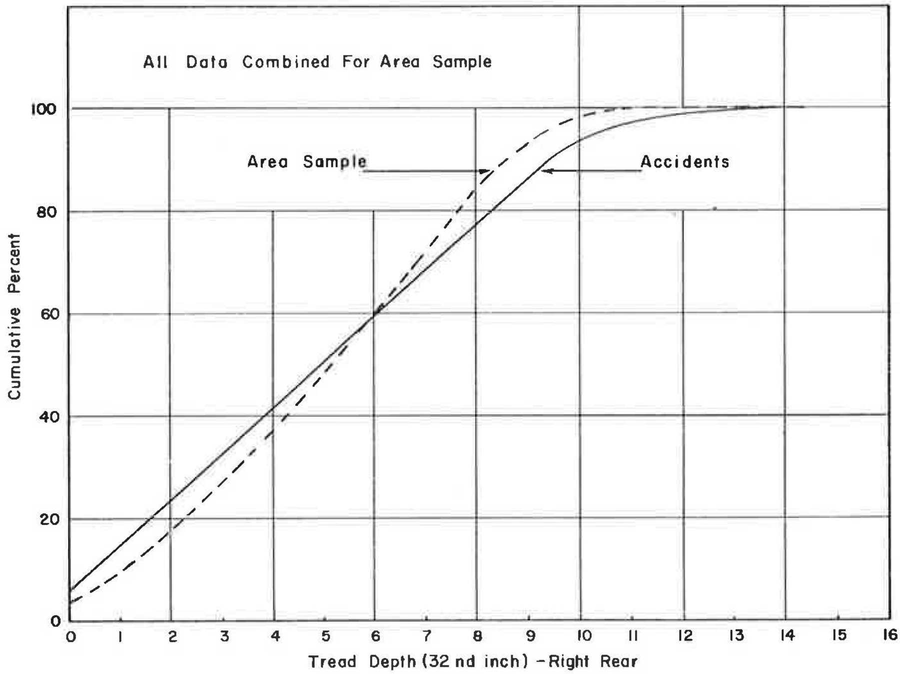

Figure 5. Comparison of right rear tire tread depths.

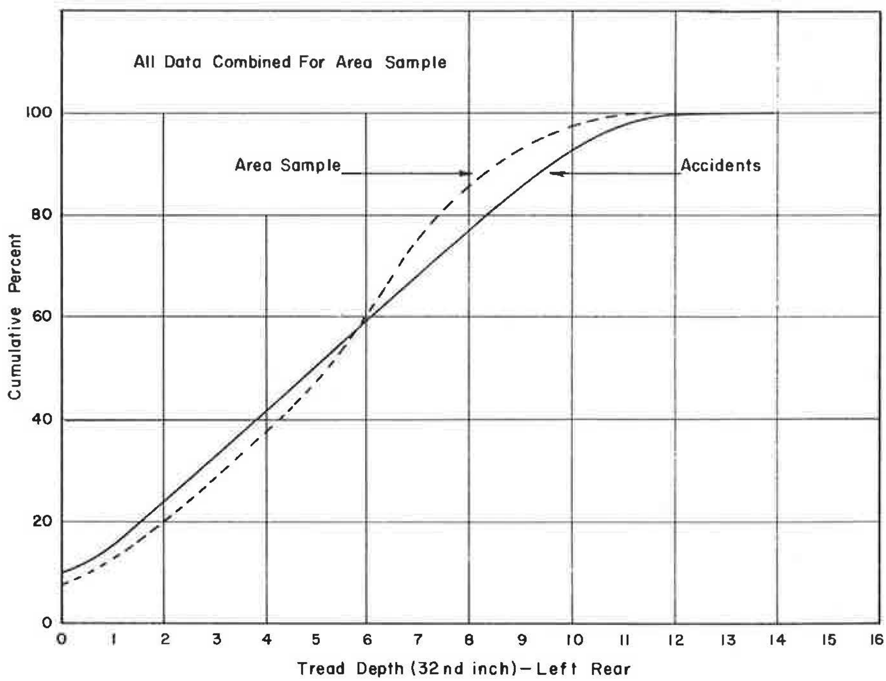

Figure 6. Comparison of left rear tire tread depths.

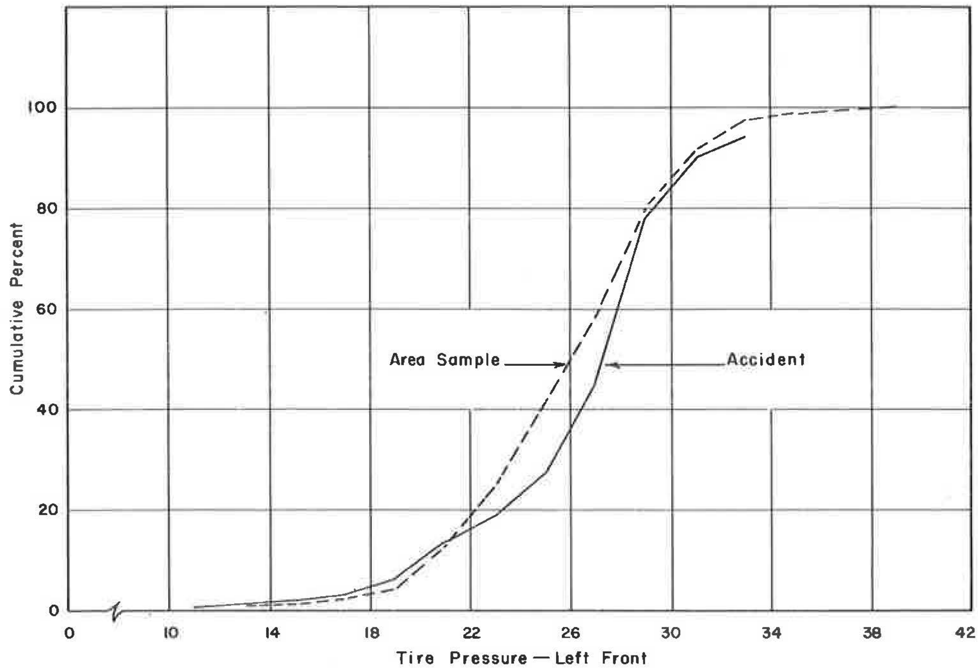

Figure 7. Comparison of left front tire pressures.

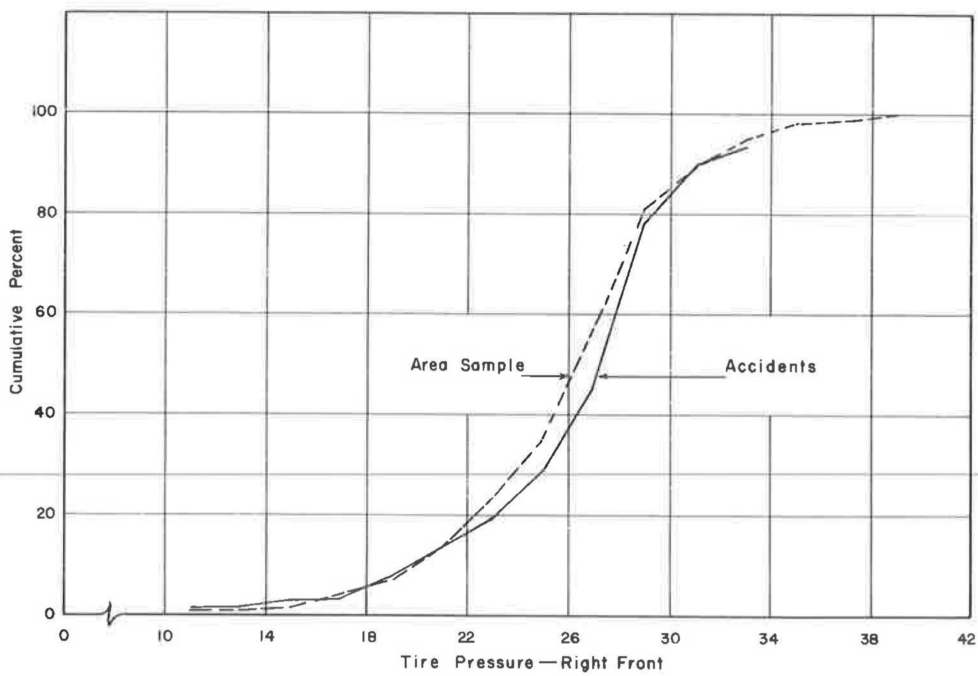

Figure 8. Comparison of right front tire pressures.

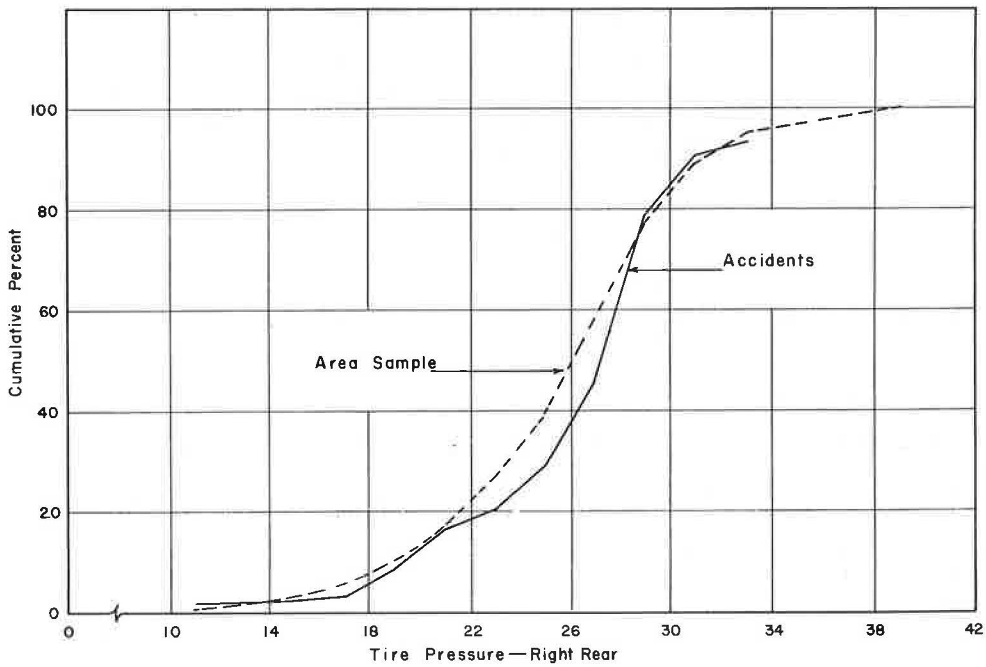

Figure 9. Comparison of right rear tire pressures.

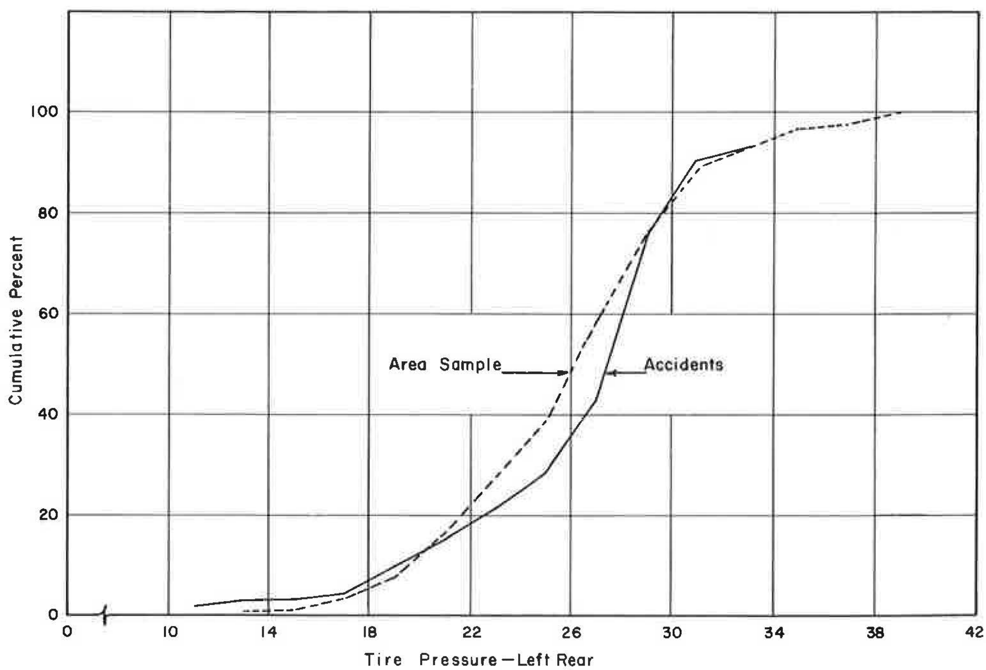

Figure 10. Comparison of left rear tire pressures.

is 36; therefore, it is concluded that there is no significant difference between skid numbers of the accident site and the 1-mile vicinity, or between the accident site and area sample. The skid numbers in Figure 11 were obtained at the accident site; the accidents generally occurred on pavements with lower skid numbers compared with the area sample.

Perhaps the most striking data collected were those of surface texture. Texas had never before obtained the quantity of texture readings collected in this project, and the distribution of data caused immediate concern. The data indicated that a very large percentage of wet-weather accidents occurred on pavement surfaces with small macrotexture values. The cumulative frequency distribution of daily vehicle-miles of travel for each texture grouping is shown in Figure 12 together with the cumulative frequency data for the accident sites. There is a large difference in percentages in the lower texture values. Near the 0.1 texture group, it appears that 40 percent of the accidents occurred on 10 to 15 percent of the pavements.

Classification of Accidents

Research personnel are sometimes accused of dividing data into increasingly smaller groups until a point can be satisfactorily supported. This may seem true at times, but here the data were believed to require certain groupings and the exclusion of some accidents in order to avoid bias. Accidents involving intoxicated drivers were removed from consideration, and vehicles with more than four tires were not included. If a multivehicle accident occurred that involved a vehicle with more than four tires, this vehicle was omitted from study because of the obviously large tire pressure and tread depth considerations; however, the other four-wheeled vehicles involved were included. There remained a total of 396 accidents, involving 540 vehicles.

The categories of accidents were arbitrarily selected, but it was believed that categories such as hydroplaning, stopping friction, and cornering friction would be particularly useful because these are amenable to remedial action and seem much more pertinent to the engineer than do the usual categories such as fatal or personal injury

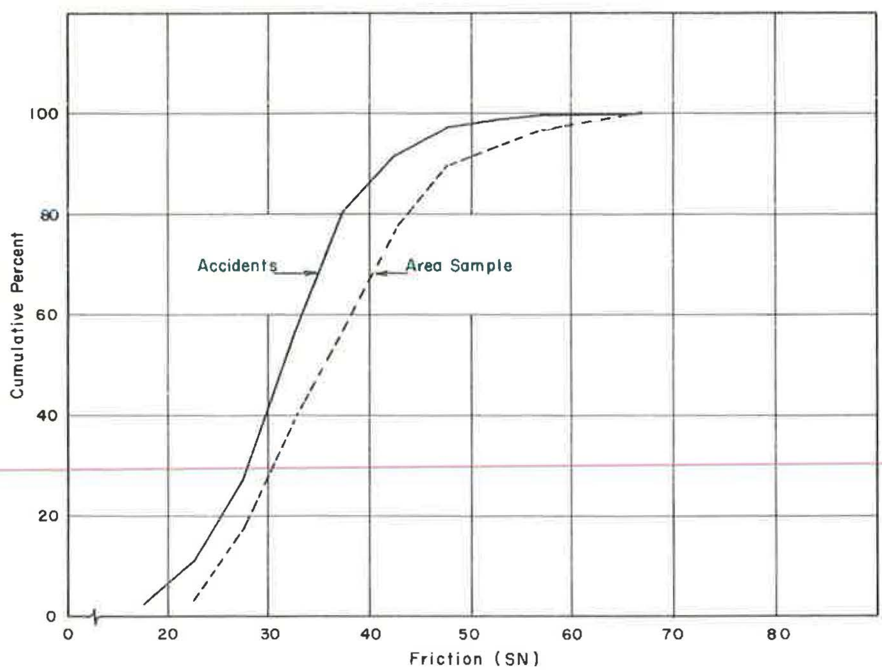

Figure 11. Comparison of friction values.

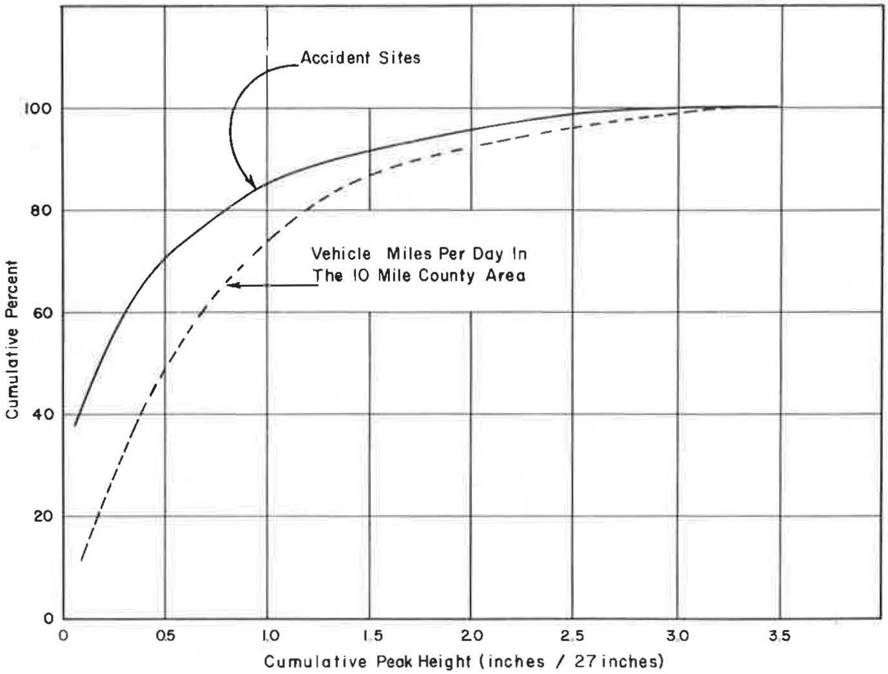

Figure 12. Comparison of macrotexture.

accident. Also, several research projects were planned for the skidding-accident area, and determination of the urgency of needed information for each friction type to establish priority ratings was required. Table 1 gives the categories into which the accidents were classified.

Originally the accidents occurring on curves were divided into those that involved a vehicle skidding to the inside of the curve, toward the radius point, and those skidding to the outside. However, little information related to the accident was provided, and the number of accidents involving skidding inside the curve was approximately equal to those skidding outside. In a large number of cases it appeared that the rear end lost traction, which caused the vehicle to spin around. Therefore, all accidents occurring on curves were included in category 2.

Categories 1, 2, 3, and 5 are thought to be closely associated with a skidding or out-of-control vehicle and tent to be single-vehicle accidents. Categories 2 and 5 generally

TABLE 1

CLASSIFICATION OF ACCIDENTS

Category		Accidents	
Description	Number	Number	Percent
Accidents occurring on a tangent or straight roadway section with no braking involved	1	57	14.4
Accidents occurring on curves	2	125	31.5
Accidents occurring on a tangent with braking involved	3	42	10.6
Accidents involving multiple vehicles	4	56	14.2
Accidents occurring while passing	5	28	7.1
Miscellaneous accidents	6	88	22.2
Accidents in categories 1, 2, 3, and 5	7	252	63.6
Accidents in all categories	8	396	100.0

involve some measure of cornering friction, category 3 involves braking friction, and category 1 involves a drive friction mode, with skidding believed to be initiated near the hydroplaning point.

Category 4 accidents include many rear-end collisions, in which at least one vehicle was moving much slower than the other vehicles involved. Category 6 accidents are those that did not fit any other specific type. (Categories 7 and 8 are discussed later.)

Each accident was classified according to the decision of the researcher. A duplicate set of accidents was classified by another member of the staff. Conflicts in categories were reviewed by both staff members and a final decision was reached. The number of accidents in each category is also given in Table 1.

The data given in Table 1 show that 25 percent of the accidents studied were single-vehicle accidents occurring on tangents. In many instances, the records of accidents in category 1 contained officers' notes that indicated the possible presence of hydroplaning and slick tires. Some 38.6 percent of the accidents involved turning or cornering manuevers (categories 2 and 5), and 14.2 percent were multivehicle. Five multivehicle accidents occurred on curves but were retained in category 4. Finally, 22.2 percent were placed in the miscellaneous category. These were accidents about which no decision could be reached concerning the grouping; for example, an accident was reported in which the vehicle was washed down a stream while the driver was attempting to negotiate a low water crossing.

Analysis of Accidents

The advantage of studying accidents by category became immediately apparent. Large differences were found in each of the five variables for accidents in a given category as compared to the study of all accidents. For example, during the course of this project it was desired to study the data concerning tread depths. Many states require tires to exhibit $^2/_{32}$-in. tread depth at the time of vehicular inspection. It was, therefore, decided to determine the percentages of vehicles with tire tread depths of $^2/_{32}$ in. or less and compare the accident vehicle sample with the area sample, or the average vehicle in the study area.

It was found that only 7 percent of the vehicles in the study area had tire tread depths of $^2/_{32}$ in. or less on the front tires. Some 8 to 9 percent of the accident vehicles had tires with $^2/_{32}$-in. tire tread depths or less. This would indicate that there is not much difference between the average vehicle and the accident vehicle; however, the rear tires caused concern because almost 25 percent of the vehicles that had an accident on wet pavement in the study area also had tires with $^2/_{32}$-in. tread depths or less. In comparison, the rear-tire tread depth of the average vehicle appears much better, with 13 to 16 percent of vehicles meeting the requirement.

The tire tread depths of the accident vehicles were also studied. The percentage of accident vehicles with $^2/_{32}$-in. or less tire-tread depth was as follows:

Category	Left Front	Right Front	Right Rear	Left Rear
1	19	22	54	52
2	12	15	38	35
3	12	19	26	23
4	8	5	5	11
5	7	21	32	46

More than 50 percent of the vehicles that had wet-weather accidents on a tangent with no braking also had rear tires with tread depths of $^2/_{32}$ in. or less. In comparison, the multivehicle accidents (predominately where one vehicle had slowed or stopped on the pavement) had tires with strikingly greater tread depths.

The rolling friction mode found in category 1 accidents requires less friction to maintain a vehicle in a selected path than does an accident related to any other friction mode. Yet this friction was not available or the accident would not have occurred. The fact that the percentages of vehicles are ordered as previously shown is considered significant in that (a) tread depth is a factor to be considered in the friction of the tire-pavement interface; (b) the classification of each individual accident (which was possibly biased by the selection determined by the researcher) was accomplished with some degree of success; and (c) the data reveal the necessity of tire inspection.

The remaining four variables could be treated as previously explained; however, it was decided to use a different statistical treatment to analyze the five variables in combination.

Degree of Influence of Factors by Accident Category

The five variables were studied by use of a statistical analysis of variance procedure. Each of the accident categories was studied separately and in total. The analysis was conducted in the following manner:

1. The 50 percentile value for each variable was found for the samples collected in the study area;
2. For each accident category, each accident was reviewed and each variables was placed in one of two groups, either greater than the 50 percentile value (+) or less (-);
3. The groups for each variable were combined into tables similar to the summary given in Table 2; and
4. An analysis of variance study of the collected data was conducted.

Two addtional accident categories were defined for this part of the study. Category 7 consists of cumulative information for categories 1, 2, 3, and 5, all of which were believed to be closely associated with a skidding (out-of-control) vehicle. Category 8 represents all accidents classified, that is, those involving four-wheeled vehicles and sober drivers.

The 50 percentile values of the samples were used because the samples represent "that which was available to the driver." Any percentile point could have been selected for study, but the 50 percentile value represented the midway point. After finding the number of accidents that occurred when the variable studied was greater or less than the established 50 percentile point, it was possible to estimate the influence of the variable. Also, combining the variables made it possible to estimate the influence of a combination of variables. It should be noted that the tire pressures and tread depths for the four wheel positions of a given vehicle were averaged for this study.

The results of the 50 percentile grouping are given in Table 2. The results are indicated for each accident category where both the number of accidents and the percentage represented by the number may be found. The numbers in the last column were obtained by adding the numbers in the preceding columns. For example, in the first row, it may be found that 13 of the category 1 accidents occurred where speed and tire pressures were greater than the 50 percentile value and where tread depths, friction (SN at 50 mph), and pavement texture are less than the 50 percentile values. These 13 accidents represent 22.7 percent of the 57 category 1 accidents. In this method of analysis, the accident sites and accident vehicles are compared to the sample pavements and vehicles that represent the normal driving condition.

By using the information given in Table 2, we can analyze individual variables; that is, the number of accidents involving a positive variable (one with a greater than 50 percentile value) may be summed and compared to the sum of the negative variable (less than 50 percentile value). Table 3 gives the results of this analysis. The percentages indicate the variables that were present in most accidents in each category. The order in which they were present is as follows:

Category	Variable Order
1	-TX, +SP, -TD, -FR, +PR
2	-TD, -FR, +PR, +SP, -TX
3	-FR, +PR, -TX, +SP, -TD
4	-SP, +TD, -FR, +PR, -TX
5	-TX, +SP, -PR, -TD, -FR
6	+PR, +TD, -TX, -SP, FR
8	-TD, -FR, +PR, -TD, +SP

TABLE 2

RESULTS OF VARIABLE ANALYSIS

Variables					Category 1		Category 2		Category 3		Category 4		Category 5		Category 6		Category 7		Category 8	
					Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
+SP	+PR	+TD	+FR	+TX	2	3.5	2	1.6	0	0	1	1.8	0	0	5	5.8	4	1.6	10	2.5
				-TX	2	3.5	3	2.4	0	0	0	0	0	0	4	4.5	5	2.0	9	2.3
			-FR	+TX	0	0	6	4.8	2	4.8	2	3.6	1	3.6	2	2.3	9	3.6	13	3.3
				-TX	6	10.4	5	4.0	6	14.2	4	7.1	1	3.6	4	4.5	18	7.0	26	6.6
		-TD	+FR	+TX	1	1.8	4	3.2	0	0	0	0	0	0	1	1.1	5	2.0	6	1.5
				-TX	4	7.0	2	1.6	2	4.8	0	0	3	10.7	4	4.5	11	4.4	15	3.8
			-FR	+TX	2	3.5	4	3.2	2	9.4	1	1.8	0	0	2	2.3	8	3.2	11	2.8
				-TX	13	22.7	19	15.2	4	9.4	0	0	0	0	4	4.5	36	14.1	40	10.0
	-PR	+TD	+FR	+TX	0	0	1	0.8	2	4.8	0	0	0	0	4	4.5	3	1.2	7	1.8
				-TX	0	0	2	1.6	0	0	0	0	1	3.6	1	1.1	3	1.2	4	1.0
			-FR	+TX	0	0	4	3.2	0	0	0	0	1	3.6	1	1.1	5	2.0	6	1.5
				-TX	2	3.5	1	0.8	2	4.8	3	5.4	1	3.6	2	2.3	6	2.4	11	2.8
		-TD	+FR	+TX	1	1.8	5	4.0	0	0	1	1.8	2	7.1	3	3.4	8	3.2	12	3.0
				-TX	3	5.3	7	5.6	1	2.4	0	0	2	7.1	0	0	13	5.2	13	3.3
			-FR	+TX	1	1.8	5	4.0	0	0	0	0	2	7.1	2	2.3	8	3.2	10	2.5
				-TX	7	12.2	5	4.0	1	2.4	0	0	5	17.9	1	1.1	18	7.0	19	4.8
-SP	+PR	+TD	+FR	+TX	0	0	1	0.8	1	2.4	3	5.4	0	0	6	6.9	2	0.8	11	2.8
				-TX	0	0	3	2.4	3	7.1	4	7.1	0	0	6	6.9	6	2.4	16	4.0
			-FR	+TX	0	0	5	4.0	2	4.8	4	7.1	0	0	5	5.8	7	2.8	16	4.0
				-TX	0	0	2	1.6	1	2.4	11	19.5	2	7.1	4	4.5	5	2.0	20	5.1
		-TD	+FR	+TX	0	0	6	4.8	0	0	1	1.8	0	0	1	1.1	6	2.4	8	2.0
				-TX	1	1.8	5	4.0	1	2.4	0	0	2	7.1	3	3.4	9	3.6	12	3.0
			+FR	+TX	1	1.8	4	3.2	2	4.8	2	3.6	0	0	4	4.5	7	2.8	13	3.3
				-TX	3	5.3	5	4.0	4	9.4	4	7.1	0	0	6	6.9	12	4.8	22	5.6
	-PR	+TD	+FR	+TX	1	1.8	0	0	0	0	0	0	0	0	0	0	1	0.4	1	0.3
				-TX	2	3.5	2	1.6	0	0	2	3.6	0	0	3	3.4	4	1.6	9	2.3
			-FR	+TX	0	0	0	0	1	2.4	2	3.6	0	0	1	1.1	1	0.4	4	1.0
				-TX	1	1.8	2	1.6	0	0	5	8.9	1	3.6	3	3.4	4	1.6	12	3.0
		-TD	+FR	+TX	0	0	2	1.6	0	0	1	1.8	0	0	0	0	2	0.8	3	0.8
				-TX	2	3.5	3	2.4	1	2.4	2	3.6	1	3.6	3	3.4	7	2.8	12	3.0
			-FR	+TX	0	0	4	3.2	1	2.4	2	3.6	0	0	1	1.1	5	2.0	8	2.0
				-TX	2	3.5	6	4.8	3	7.1	1	1.8	3	10.7	2	2.3	14	5.5	17	4.3
Total					57	100.0	125	100.0	42	100.0	56	100.0	28	100.0	88	100.0	252	100.0	396	100.0

TABLE 3

RESULTS OF SINGLE VARIABLE ANALYSIS

Variable	Category 1		Category 2		Category 3		Category 4		Category 5		Category 6		Category 8	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
+SP	44	77	75	60	22	52	12	21	19	68	40	46	212	54
-SP	13	23	50	40	20	48	44	79	9	32	48	54	184	46
+PR	35	61	76	61	30	71	37	66	9	32	61	69	248	63
-PR	22	39	49	39	12	29	19	34	19	68	27	31	148	37
+TD	16	28	39	28	20	48	41	73	10	36	51	58	175	44
-TD	41	72	90	72	22	52	15	27	18	64	37	42	221	56
+FR	19	33	48	38	11	26	15	27	11	39	44	50	148	37
-FR	38	67	77	62	31	74	41	73	17	61	44	50	248	63
+TX	9	16	53	48	13	31	20	36	6	21	38	43	139	35
-TX	48	84	72	58	29	69	36	64	22	79	50	57	257	65

Variables for categories 1, 2, and 3 are similar to those for category 8; that is, the large percentages of accidents occurred at high speeds, high tire pressures, small tread depths, low friction, and low textures. Variables for category 5 were similar to those for categories 8, 1, 2, and 3 except for tire pressure, which was below the 50 percentile range in category 5.

High number of accidents seem to appear randomly in the cells of Table 2, but closer study shows that there is a pattern. As an example, the largest number for category 1 accidents was 13 (22.7), which reflected high speed, high pressure, small tread depths, low friction, and small textures. The largest number for category 2 occurred with the same combination of variables. This kind of pattern is believed to result from the interaction of variables. Generally, every fourth cell, observed horizontally across the table, has a high number of accidents. This pattern shows the importance and danger of the combination of three variables: small tread depths, low friction, and small textures. However, based on these data, there would not be much danger from a category 1 accident if low speed, high pressure, and good tread depths were maintained because none of the 57 accidents studied occurred under these conditions.

Probably the best way to analyze the interaction of variables is through the use of the statistical analysis of variance procedure. The data given in Table 2 actually conform to a 2^5 factorial, which can be easily used in a computer program developed for this purpose. The computer program selected was the step-wise regression program developed at the University of California (5). The program calculates the sum of squares for each variable or combination of variables studied. For a factorially designed experiment, the sum of squares value can be used to reveal the amount each variable or combination of variables explains, or contributes to, the number of accidents found in the study. By accumulating the sum of squares values for each variable, a total sum of squares can be determined. Because we wanted to determine the significance of each variable or combination of variables, the sum of squares for each individual variable was divided by the total sum of squares to determine the percentage of contribution. These percentages are given in Tables 4 through 11.

An inadequacy of the method of analysis, i.e., the small number of observations or accidents available for study, should be noted. There were five original variables and 26 other possible combinations of the original variables. Including the number of accidents as the variable for study (the dependent variable), there is a total of 32 variables.

This inadequacy was severe for category 5 (Table 8), for which there were only 28 accidents or observations. Here the attempt to solve for the contribution of 32 variables was made by using the distribution of only 28 accidents. As an indication of what to accept and what not to accept, the authors arbitrarily selected as significant any analysis of variance study containing at least 32 observations.

It is obvious from the data given in the tables that each of the five variables is an important contributor to the accidents studied and that some are more important to certain

TABLE 4

CONTRIBUTION OF VARIABLES TO CATEGORY 1 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
TX	47.5	21.5	21.5
SP	30.1	13.6	35.1
TD	19.5	8.8	43.9
SP, TX	13.8	6.1	50.0
FR, TX	13.8	6.1	56.1
FR	11.2	5.1	61.2
SP, PR	11.3	5.1	66.3
TD, TX	11.3	5.1	71.4
SP, FR, TX	11.3	5.1	76.5
SP, FR	9.0	4.1	80.6
SP, TD	7.0	3.2	83.8
TD, FR	7.1	3.2	87.0
PR	5.3	2.4	89.4
SP, PR, TX	5.2	2.4	91.8
PR, FR	3.8	1.7	93.5
SP, TD, TX	2.5	1.1	94.6
PR, FR, TX	2.6	1.1	95.7
PR, TX	1.5	0.7	96.4
PR, TD, FR	1.5	0.7	97.1
SP, PR, FR, TX	1.6	0.7	97.8
PR, TD	0.8	0.4	98.2
SP, PR, TD	0.7	0.4	98.6
SP, TD, FR	0.8	0.4	99.0
TD, FR, TX	0.8	0.4	99.4
SP, PR, TD, FR	0.8	0.4	99.8
PR, TD, TX	0.3	0.1	99.9
SP, TD, FR, TX	0.2	0.1	100.0
SP, PR, FR	0.1		
SP, PR, TD, TX			
PR, TD, FR, TX			
SP, PR, TD, FR, TX			
Total	221.5	100.0	

TABLE 5

CONTRIBUTION OF VARIABLES TO CATEGORY 2 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
TD	69.0	20.2	20.2
FR	26.3	7.8	28.0
TD, FR, TX	26.3	7.8	35.8
PR	22.8	6.8	42.6
SP, PR, FR	22.8	6.8	49.4
SP	19.5	5.8	55.2
SP, PR, TD, FR	19.5	5.8	61.0
SP, PR, FR, TX	19.5	5.8	66.8
PR, TD, FR, TX	16.6	4.8	71.6
SP, PR, TX	13.8	4.0	75.6
TX	11.2	3.3	78.9
PR, FR	11.3	3.3	82.2
SP, FR	9.0	2.6	84.8
TD, TX	9.1	2.6	87.4
SP, TD, TX	9.0	2.6	90.0
PR, FR, TX	9.0	2.6	92.6
SP, TD, FR, TX	5.3	1.6	94.2
PR, TD, TX	3.8	1.1	95.3
SP, PR, TD, FR, TX	3.8	1.1	96.4
SP, TD	1.5	0.5	96.9
SP, TX	1.5	0.5	97.4
TD, FR	1.6	0.5	97.9
FR, TX	1.5	0.5	98.4
SP, TD, FR	1.5	0.5	98.9
SP, PR, TD, TX	1.6	0.5	99.4
PR, TX	0.8	0.2	99.6
PR, TD, FR	0.7	0.2	99.8
SP, PR	0.3	0.1	99.9
PR, TD	0.3	0.1	100.0
SP, PR, TD			
SP, FR, TX			
Total	342.7	100.0	

TABLE 6

CONTRIBUTION OF VARIABLES TO CATEGORY 3 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
FR	12.5	18.7	18.7
PR	10.1	15.2	33.9
TX	8.0	12.1	46.0
SP, TD, FR, TX	6.2	9.4	55.4
PR, FR	4.5	6.8	62.2
SP, PR, FR	4.5	6.8	69.0
SP, TD, FR	4.5	6.8	75.8
SP, FR, TX	3.1	4.6	80.4
PR, TX	2.0	3.0	83.4
TD, TX	2.0	3.0	86.4
SP, PR, TD, FR	2.0	3.0	89.4
SP, TD	1.1	1.6	91.0
FR, TX	1.1	1.6	92.6
SP, PR, TD	1.1	1.6	94.2
SP, FR	0.6	0.9	95.1
SP, TX	0.5	0.7	95.8
TD, FR	0.5	0.7	96.5
SP, PR, TX	0.4	0.6	97.1
SP, TD, TX	0.5	0.7	97.8
PR, TD, TX	0.6	0.9	98.7
SP	0.1	0.1	98.8
TD	0.1	0.1	98.9
SP, PR	0.1	0.1	99.0
PR, TD	0.2	0.3	99.3
PR, FR, TX	0.1	0.1	99.4
TD, FR, TX	0.1	0.1	99.5
SP, PR, FR, TX	0.1	0.1	99.6
PR, TD, FR, TX	0.2	0.3	99.9
SP, PR, TD, FR, TX	0.1	0.1	100.0
PR, TD, FR			
SP, PR, TD, TX			
Total	66.9	100.0	

TABLE 7

CONTRIBUTION OF VARIABLES TO CATEGORY 4 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
SP	32.0	20.0	20.0
TD	21.1	13.1	33.1
FR	21.2	13.1	46.2
TD, TX	10.1	6.3	52.5
PR	10.1	6.3	58.8
TX	8.0	5.0	63.8
PR, TD	8.0	5.0	68.8
TD, FR	8.0	5.0	73.8
FR, TX	6.1	3.8	77.6
SP, TX	4.5	2.8	80.4
PR, FR	4.5	2.8	83.2
TD, FR, TX	4.5	2.8	86.0
SP, PR, FR, TX	4.5	2.8	88.8
SP, PR	3.2	2.0	90.8
SP, TD	3.1	2.0	92.8
SP, FR	3.1	2.0	94.8
SP, PR, TD	2.0	1.3	96.1
PR, FR, TX	2.0	1.3	97.4
SP, PR, TX	1.1	0.7	98.1
SP, TD, TX	1.2	0.7	98.8
SP, PR, FR	0.5	0.3	99.1
SP, PR, TD, TX	0.5	0.3	99.4
PR, TX	0.1	0.1	99.5
SP, FR, TX	0.1	0.1	99.6
PR, TD, FR	0.1	0.1	99.7
SP, PR, TD, FR	0.2	0.1	99.8
PR, TD, FR, TX	0.1	0.1	99.9
SP, PR, TD, FR, TX	0.1	0.1	100.0
SP, TD, FR	0.0		
PR, TD, TX	0.0		
SP, TD, FR, TX			
Total	160.0	100.0	

TABLE 8

CONTRIBUTION OF VARIABLES TO CATEGORY 5 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
TX	8.0	17.7	17.7
TD	4.5	9.9	27.6
PR, TD, FR	4.5	9.9	37.5
PR, TD, FR, TX	4.5	9.9	47.4
SP	3.1	6.9	54.3
PR	3.2	6.9	61.2
PR, TD	3.1	6.9	68.1
SP, PR	2.0	4.5	72.6
PR, FR	2.0	4.5	77.1
TD, TX	2.0	4.5	81.6
PR, FR, TX	2.0	4.5	86.1
FR	1.1	2.5	88.6
SP, TD	1.1	2.5	91.1
TD, FR	1.1	2.5	93.6
SP, PR, TD	0.6	1.1	94.7
SP, FR, TX	0.5	1.1	95.8
SP, PR, TD, TX	0.5	1.1	96.9
SP, TD, FR, TX	0.5	1.1	98.0
SP, TX	0.1	0.2	98.2
PR, TX	0.1	0.2	98.4
FR, TX	0.1	0.2	98.6
SP, PR, FR	0.2	0.2	98.8
SP, TD, TX	0.1	0.2	99.0
PR, TD, TX	0.1	0.2	99.2
TD, FR, TX	0.1	0.2	99.4
SP, PR, TD, FR	0.2	0.2	99.6
SP, PR, FR, TX	0.1	0.2	99.8
SP, PR, TD, FR, TX	0.1	0.2	100.0
SP, FR			
SP, PR, TX			
SP, TD, FR			
Total	45.5	100.0	

TABLE 9

CONTRIBUTION OF VARIABLES TO CATEGORY 6 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
PR	36.1	36.1	36.1
SP, PR, TX	10.1	10.1	46.2
TD	6.2	6.2	52.4
TD, FR	6.1	6.1	58.5
TX	4.5	4.5	63.0
SP, TX	4.5	4.5	67.5
SP, FR, TX	4.5	4.5	72.0
PR, TD, FR	4.5	4.5	76.5
PR, TD, TX	5.0	5.0	81.5
PS, PR	2.6	2.6	84.1
SP	2.0	2.0	86.1
SP, FR	2.0	2.0	88.1
PR, TD	2.0	2.0	90.1
SP, PR, TD, FR	2.0	2.0	92.1
PR, TX	1.2	1.2	93.3
TD, TX	1.1	1.1	94.4
TD, FR, TX	1.1	1.1	95.5
SP, PR, FR, TX	1.1	1.1	96.6
SP, TD, FR, TX	1.2	1.2	97.8
FR, TX	0.5	0.5	98.3
SP, PR, TD	0.5	0.5	98.8
SP, PR, TD, FR, TX	0.5	0.5	99.3
SP, TD	0.1	0.1	99.4
PR, FR	0.1	0.1	99.5
SP, PR, FR	0.1	0.1	99.6
SP, TD, FR	0.2	0.2	99.8
SP, TD, TX	0.1	0.1	99.9
PR, FR, TX	0.1	0.1	100.0
FR			
SP, PR, TD, TX			
PR, TD, FR, TX			
Total	100.0	100.0	

TABLE 10

CONTRIBUTION OF VARIABLES TO CATEGORY 7 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
TX	253.1	18.2	18.2
TD	231.2	16.6	34.8
FR	171.1	12.2	47.0
SP	144.4	10.3	57.3
TD, TX	84.6	6.0	63.3
PR	72.0	5.1	68.4
SP, FR	45.1	3.2	71.6
SP, PR, FR	45.1	3.2	74.8
SP, PR, TX	45.1	3.2	78.0
FR, TX	40.5	2.9	80.9
SP, FR, TX	40.5	2.9	83.8
PR, FR	36.2	2.6	86.4
SP, PR, FR, TX	32.0	2.3	88.7
SP, TX	28.1	2.0	90.7
TD, FR, TX	28.1	2.0	92.7
SP, TD, TX	18.0	1.3	94.0
SP, TD	15.1	1.1	95.1
TD, FR	12.5	0.9	96.0
SP, PR, TD, FR	12.5	0.9	96.9
PR, TX	10.2	0.7	97.6
SP, PR	8.0	0.6	98.2
PR, FR, TX	8.0	0.6	98.8
PR, TD, FR, TX	6.1	0.4	99.2
SP, PR, TD, TX	4.5	0.3	99.5
PR, TD	2.9	0.2	99.7
PR, TD, TX	2.0	0.1	99.8
SP, TD, FR, TX	1.1	0.1	99.9
SP, PR, TD, FR, TX	1.2	0.1	100.0
SP, PR, TD	0.1		
SP, TD, FR	0.0		
PR, TD, FR			
Total	1,399.5	100.0	

TABLE 11

CONTRIBUTION OF VARIABLES TO CATEGORY 8 ACCIDENTS

Variable	Sum of Squares	Percent	Cumulative Percent
TX	435.1	24.9	24.9
PR	312.5	17.9	42.8
FR	312.5	17.9	60.7
FR, TX	91.2	5.2	65.9
SP, PR, TX	78.1	4.4	70.3
PR, FR	72.0	4.1	74.4
TD	66.1	3.8	78.2
SP, FR, TD	66.1	3.8	82.0
TD, TX	50.0	2.9	84.9
SP, TD	36.1	2.0	86.9
PR, TD	36.2	2.0	88.9
SP, PR, FR	32.0	1.8	90.7
SP	24.5	1.4	92.1
SP, TD, TX	24.5	1.4	93.5
PR, TX	21.1	1.3	94.8
PR, FR, TX	51.1	0.8	95.6
SP, FR	12.5	0.7	96.3
PR, TD, TX	12.5	0.7	97.0
TD, FR	10.1	0.6	97.6
SP, PR, TD, TX	8.0	0.5	98.1
PR, TD, FR	6.2	0.4	98.5
SP, PR, FR, TX	6.1	0.4	98.9
TD, FR, TX	4.5	0.3	99.2
PR, TD, FR, TX	4.5	0.3	99.5
SP, PR, TD	3.1	0.2	99.7
SP, PR, TD, FR	3.1	0.2	99.9
SP, TX	1.2	0.1	100.0
SP, PR	0.5		
SP, TD, FR	0.1		
SP, TD, FR, TX	0.0		
SP, PR, TD, FR, TX			
Total	1,745.5	100.0	

types of accidents than are others. For example, the more complex interactions of four and five variables are not so important as the less complex interactions of two and three variables. In each case, a single variable is most important. Category 2 and category 6 accidents appear to be the most complex because of the more complex interactions. It is possible that this complexity is associated with the miscellaneous nature of category 6 accidents and with the variables selected for measurement in the category 2 accidents. For example, the cornering slip friction may not be directly related to the pavement surface friction (SN at 50 mph).

CONCLUSIONS

Macrotexture, vehicle tread depth, pavement surface friction (SN at 50 mph), vehicle speed, and vehicle tire pressure were found to be important variables in the wet-weather accidents studied.

Compared to the sample data, the accident data indicated that a larger number of accidents occurred under the following conditions:

1. The texture of the pavement at the accident site was small (or fine macrotexture);
2. The tread depths of the vehicle involved were small;
3. The friction value of the pavement at the accident site was low;
4. The speed of the vehicle immediately prior to the accident was high; and
5. The tire pressures of the accident vehicle were high.

The relative importance of these five variables was found to depend on the type of general accident situation.

Single variables were the most significant contributors to wet-weather accidents. Complex interactions of the five variables studied did not influence accidents to a great degree, but several interactions of two or three contributed significantly.

Approximately 40 percent of the vehicles involved in wet-weather accidents were, at the time of the accident, in a turning maneuver, about 33 percent were on horizontal curves, and 7.1 percent were passing. It is apparent that research efforts should be directed toward obtaining more information on cornering friction and that remedial measures should be directed to horizontal curves.

For a skidding accident in wet weather (category 7), the order of importance of the five variables studied was (a) texture, (b) tread depth, (c) friction, (d) speed, and (e) tire pressure. Little difference was found in the percentage of contribution of each variable, with values ranging from 5.1 to 18.2 percent.

One of the purposes of collecting accident information should be to detect trouble areas in order that remedial measures may be developed. It would seem to the design engineer that the sole purpose of maintaining accident records is to determine the number of people killed or injured per year in order to make annual comparisons. The results of this report indicated that close study of the reports of the investigating officers should help to classify accidents into preselected categories. It is believed that the classification of wet-weather accidents into categories that contain the various friction modes would be of benefit because these friction modes are also used in highway design. It is concluded that accident information should also be reported in terms useful to the engineer.

The most striking result of the study was the importance of texture. As stated previously, few texture measurements had been obtained prior to this project, and the first indication of the effect of texture was the distribution of texture values (Fig. 12). When texture at the accident site was compared to the texture available on the roadways in the area, an even greater importance was indicated. The texture sampling procedure for the area was poor because funds sufficient to sample the study area had not been provided, but the evidence accumulated appeared to offer cause for concern.

It should be noted that macrotexture of the surface itself is not the item of interest; the minute water drainage channels that texture provides when a tire passes over it are the significant effects. A porous surface through which the water could drain when the tire passes (or under its own head) would be as beneficial, but a textured surface would probably be the most economical type of construction in Texas.

The distribution of accidents given in Tables 2 and 3 is considered to be significant and to show the benefit of studying accidents by categories. Except for tire pressures, the distribution is as expected. The most dangerous combination of variables is high speeds, low tread depths, low friction, and small textures. Each of these variables helps to bring about a very low available friction between the tire and the pavement under wet-weather conditions.

The absence of low tire pressures in the accidents is puzzling. According to hydroplaning theory, low tire pressures are directly related to the velocity of dynamic hydroplaning; that is, the lower the tire pressure is, the lower the speed is at which hydroplaning occurs (6, 7). When a vehicle is hydroplaning, the available tire-pavement friction must be very low and, prior to this study, it would have seemed that all proven principles associated with hydroplaning would also hold for tire-pavement friction. Yet in only one accident type, passing, were low tire pressures found to be significant. Apparently, the absence of low tire pressures from the group of significant variables can be explained by viscous hydroplaning, which is generally associated with slick pavements, small water depths, and high tire pressures. It is possible that both viscous and dynamic hydroplaning are involved in these accidents, and the two could not be separated with the methods used in this analysis. However, it does appear that the accidents occurring because of hydroplaning cannot be separated from accidents occurring at a low friction level. In other words, the situation is dangerous whenever friction is reduced to a low level. It is obvious that the pavement is not the only contributor to low friction. Low tread depths and high speeds also contribute to an accident. A good measure of water film depth on the pavement at the time of the accident was not obtained in this study, but all the evidence when combined with theory leads to the importance of this variable. Water depths greater than those emitted by the skid trailer must be present at the time of the accident.

Accidents are complex, and in many cases no remedial measure is available as far as design, construction, and maintenance of highways or enforcement of laws are concerned. Highway departments and law enforcement agencies can do much to reduce accidents, but the evidence in this study indicates that the driver must also act independently to prevent the accidents analyzed here.

The following items are suggested for implementation. Ways to obtain sufficient texture should be considered in highway design, construction, and maintenance. A minimum texture of 0.5 in. per 27 in., measured with SWRI texturemeter, equivalent to approximately 0.035 in. by the sand patch method, is suggested for design purposes. At this value, the numbers of accidents appeared to decrease to a relatively constant value (Fig. 12). The suggested value does not provide an exceedingly coarse or harsh texture and, therefore, the high-speed friction should be optimized with road noise.

Continuing effort should be made to maintain sufficient friction on the pavement surface. Efforts being made throughout the nation to specify a nonpolishing aggregate for use in the pavement surface could be used to advantage. A method to reduce driving speeds in wet weather should be developed, and minimum tread depths should be required on vehicles that use public highways.

A skid trailer test value alone is not sufficient to establish the skidding safety of a highway or a safe friction value for a highway. However, skid trailer values must be considered in skidding safety and cannot be taken lightly. Efforts should be made to provide a tire-pavement interface friction value that is more representative of the friction available to the driver at the time of a wet-weather accident. The trailer skid number should be modified by a water depth factor and a factor for the tire-pavement drainage characteristic to give a skid number that is more representative of the actual accident conditions. The friction should be further modified by use of tread depths, tire pressures, and speeds more representative of the actual accident vehicle.

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SPOTTING SKID-PRONE SITES ON WEST VIRGINIA HIGHWAYS

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This paper describes the system developed by the West Virginia Department of Highways for identifying skid-prone sites, determining for each the differential between the actual skid number and the required skid number, programming for remedial measures in skid resistance (including resurfacing, change in superelevation, and drainage), and evaluating other features related to cause of skid, such as alignment, sight distance, and condition of pavement and shoulder. Relationships between rates of accidents in West Virginia on wet and dry pavements and various geometrics were determined. Some simplified formulas were developed for calculating required skid number in terms of deceleration rates and centrifugal force. Also discussed in the paper are the cost-effectiveness relations involved in analysis in which the average cost of an accident is taken to be about $2,000. It is assumed that, because of annual fluctuation in small samples, a reported number of three accidents at a site per year can be reduced by at least one or more per year, thus making a $2,000 investment in anti-skid measures for such a site a worthwhile investment. (An investment of $2,000 will be the approximate cost of treating intersection approaches.)

•BECAUSE of the increasing rate of wet-pavement traffic accidents throughout the United States, intensified concern and attention are being given to developing anti-skid measures. In West Virginia, there is recognition that the skidding problem is serious and can become worse. We also recognize that the problem can be reduced.

The statewide rate for reported accidents in West Virginia is currently about 400 accidents per 100 million vehicle-miles of travel. Assuming that roads are wet 15 percent of the time, the statewide wet-pavement accident rate is approximately 650 accidents per 100 million vehicle-miles. This is nearly 2.2 times the statewide accident rate on dry pavement, which is about 300 accidents per 100 million vehicle-miles.

Skid-prone sites analyzed in our study have a wet rate ranging up to 40 times the statewide average wet rate and 85 times the statewide average dry rate. Fortunately we found only one site with such an extreme rate; however, in 1970 we found more than 50 sites with a benefit-cost ratio ranging from 1.0 to 18.0. If we eliminate one wet-pavement accident in ½ mile by skid-resistant treatment, we calculate a benefit-cost ratio of about unity.

Accident rates are only one facet of the problem. A translation of rates into economics makes the problem more meaningful. The average accident cost in terms of travel is about 1 cent per vehicle-mile. At the site where the wet-accident rate was 40 times the statewide wet-accident rate, the accident cost soared to 52 cents per vehicle-mile on wet pavement.

We estimate that the average cost of an accident is about $2,000. Because wet-pavement accident costs are accumulating at the rate of 2.2 times the costs of dry-pavement accidents, we are fortunate that pavements are wet only an estimated 15 percent of the time.

Although the phenomenon of aggregate polishing in wearing surfaces has been noted in some states for 15 years or more, it has only recently begun to have any noticeable

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effect on skidding in West Virginia. This phenomenon is particulary prevalent in about 10 of the 55 counties. The problem exists on both bituminous and concrete pavements.

Skidding also results from another condition: fatty spots from rich or poorly controlled bituminous mixes. Wearing surfaces placed in extremely hot weather and opened to heavy traffic soon after rolling may tend to bleed if the bituminous content is high. Skid numbers on such surfaces may be dangerously low. We are solving this problem through quality assurance control and by special skid-resistant mixes.

Our initial idea was that poor geometrics combined with high traffic volume would help identify sections needing attention. However, we found no firm consistency in the results of our examination of about 100 sections with respect to geometrics, ADT, and the wet-pavement accident rate. We then recognized that other surface conditions and traffic characteristics peculiar to certain sites contributed to skidding.

We ran wet-dry accident ratios for night and day, for each county, for various degrees of curve, and for different pavement widths. Our analysis of a small sample indicates that the wet-dry ratios by district and geometrics are closer to 2.0 than to the statewide average value of 2.2, except for two highway districts and for 16-ft pavements and winding roads where the ratio is approximately 3.0.

We also have a problem with accident reporting. Not all of the highway traffic accidents subject to reporting are reported. Some of these are not precisely located by milepost, and some 10 percent of the accident forms do not report weather or pavement condition.

PROGRAM PURPOSE

The West Virginia program to implement anti-skid measures includes the following measures:

1. Identify sites by the various means available;
2. Examine sites and available records for various contributing causes and conditions;
3. Determine practicable corrective measures;
4. Discriminate among aggregates for feasible skid-resistant types;
5. Set minimum specifications for all wearing surfaces;
6. Attempt correlation among wet-pavement accidents, skid number, roadway condition, ADT, geometry, access, and other relevant parameters;
7. Determine upgraded skid numbers required for varying geometry, operating (or design) speeds, and ADT;
8. Determine effect of change in skid number on accident rate through before-and-after studies and long-range trend studies;
9. Reduce both the number of incidents and accidents; and
10. Develop statewide portrayal of skid-number demands for each highway section for periodic comparison with actual skid-number value to determine when decline is below critical level.

PROGRAM PHILOSOPHY

We knew at the outset that the skidding accident complex has many factors and cannot be simplified into a one-dimensional problem with f as the one independent variable. For example, in a recent study it was found that about 3 drivers in 1,000 were involved in a skidding accident on a certain slick section of highway. Why three? Few, if any, studies show the same number of accidents per 1,000 drivers, or per 100 million vehicle-miles. Accident rates also fluctuate from year to year. In 1967, we found 46 sites with a rate of 3 or more wet-pavement accidents per ½ mile. In 1969, we checked 52 sites by using the same criterion, but only 4 of the 1967- and 1969-sites were identical.

The "situation complex" that issues in an accident is measured in terms of probabilities. There is a fluctuation of from 50 to 200 percent even for 100 million vehicle-miles. Perhaps a base of a billion vehicle-miles is required for "stability."

The skidding accident complex involves tire-pavement friction interaction that is influenced by factors of instantaneous speed, acceleration or deceleration, centrifugal force, steering angle, tire type, tread wear, pavement surface texture and wear, driver behavior, rain and visibility, volume-density-speed of the traffic stream, and traffic control and regulation. Therefore, many combinations can produce an accident, and all required conditions for a skid must be present if a skid is to occur. If all required conditions except one exist, the skid will not take place. A skid-resistant surface can often provide the exception that prevents the accident. On the other hand, a wet pavement, a bare tire, a minus grade of 6 percent, a 10-deg curve with superelevation of 0.04, an instantaneous speed of 55 mph decelerating at a rate of 16 ft/sec^2, and a skid number of 30 at 40 mph will cause an accident.

Sudden changes in speed or direction induce accidents. This happens when incipient skidding exists and sliding is initiated in changing direction or speed. Whenever the kinetic forces exceed the coefficient of friction between tire and pavement, a sliding state results. This condition can happen when there is either gradual or sudden transition of pavement skid number from adequate to inadequate, sometimes caused by fatty spots. It can be made to happen with dramatic suddenness when the driver, possibly because of a change in highway geometrics, changes an adequate pavement-tire friction interaction to an inadequate one by suddenly increasing the frictional demand above the capability. In the first instance, the skid number decreases below a constant demand. In the second instance, the demand increases above a constant skid number. In some instances both conditions may happen simultaneously.

In view of this rationale, it follows that the tire-pavement interaction is the controlling feature. The highway engineer can control only the pavement texture and wear on existing highways. Another possible action of promise would be the enactment of legislation outlawing the use of bare tires or tires whose interaction with a pavement produces a skid number of less than 40. In the long run, it might prove more economical to users to retread tires than to retread pavements.

The f-requirements (drag) on level tangents seldom exceed 0.10 for steady velocity even up to 100 mph. Acceleration, except that of high-powered cars accelerating from a stop, seldom causes wheels to spin, and this can be stopped by release of the accelerator. Hence, unless there is deceleration or centrifugal force involved, tire-pavement friction is adequate and has a good safety factor (from 2 to 6). If a curve of 10 deg is introduced where the design speed is 50 mph, the skid number of 40 should be approximately 60. (A tight passing maneuver on a sharp curve can initiate a skid.) If we add a grade of 10 percent, 10 points would have to be added to the skid number. If we attempt to decelerate at ½ g (16 ft/sec^2), a skid number of 50 at 40 mph is needed. Hence, it is seen that the demands at curves, grades, intersections, and areas of free access can require the highest feasible skid number. If we can provide a coefficient of friction in excess of the brake coefficient, the brake will slip while wheels still turn. This should provide an extra measure of safety.

To summarize, wet-pavement accidents occur when tractive force exceeds tractive resistance. This can occur when deceleration rates exceed one-third of the skid number (see formula 15) or when speed and degree of curve combine to produce centrifugal force (less e) in excess of one-third of the skid number.

We checked the dry rate and wet rate on open rural level tangents to assist in our analysis and found them to be 200 and 440 respectively or a ratio of 2.2. However, the wet rate on level tangents is only 1.1 times the statewide average for all accidents and only 1.5 times the statewide dry rate. We intend to explore this facet further because we believe that, on the open rural level tangent where traffic is free flowing, the wet rate will not be substantially more than the dry rate because the demand-capability ratio (from emergency stops) should not exceed unity often.

We also asked the question: What is the urgency of corrective action? This is answered in terms of cost-effectiveness. For every skidding accident eliminated there is a savings of more than $2,000. This means that we can economically invest up to $2,000 annually for every accident eliminated. We arbitrarily set a benefit-cost ratio of at least 3 as the cutoff point. Skid-resistant treatment costs about $10,000 per mile and is estimated to retain its value for a minimum of 3 years. Then for each site analyzed the annual cost computation would be in the order of about $3,300 per mile of treatment. We have assumed that by skid-resistant treatment we can reduce both wet- and dry-skidding accidents to statewide norms. We are aware of skid number and accident relationships determined from previous studies. When we accumulate an adequate data bank on before-and-after skid numbers related to wet and dry pavements, accident reduction, and geometrics, we will try to sharpen our prognosis. We believe we can reduce wet-pavement accidents on certain highway sections by some 200 accidents annually (a savings of about $400,000) by investing about $125,000.

STRATEGY

Our strategy is to use immediately such information as we can appropriate from available sources and to engage in a long-term program to sharpen our knowledge of cost-effectiveness. Our strategy includes the following:

1. An immediate skid-trailer inventory of sites having critical geometrics with high traffic volumes to search out skid-prone sections;
2. The use of annual accident records merged with road geometrics to discover sites of excessive wet-pavement accidents, the computation of required skid numbers, and the determination of actual numbers by skid trailer (this information would be used with the skid-trailer special inventory to develop a de-slicking program);
3. A skid-trailer inventory of skid-prone sections to link skid numbers and other pertinent information to aggregate source (where aggregate polishing is the probable cause of accident);
4. A skid-resistant test road with satellites in each highway district for correlative tests over a 5-year (or longer) period to find feasible aggregates and mix designs for skid-resistant wearing surfaces;
5. A study of required skid numbers in terms of geometry, ADT, access, and speed to suggest tentative skid numbers related to highway function; and
6. A program of before-and-after and long-term studies to determine cost-effectiveness of treatments and to provide additional information for economical mix designs.

This total effort in addition to the cost of skid-resistant treatments will cost roughly $50,000 per year for the next 5 years. We admit that from this effort we may find it necessary to pay premium prices for aggregates for wearing surfaces on certain sections of highways; however, the road users who pay the premium prices will, we hope, be saving themselves many times the added user cost in reduced skidding accident costs.

DE-SLICKING PROGRAM

Reports

Skid-prone sites are reported in accident records, police alerts, district engineers' reports, news media reports, citizen complaints, and other reports. Occasionally near-accidents such as fish tailing and other skidding incidents are reported that do not result in property damage or injury. In addition, the skid-trailer is being used in a systematic coverage of the state signed system to locate sites whose actual skid numbers are less than the design speed.

Computer Program

By computer scanning we are able to locate sites having 3 or more wet-pavement accidents for each ½-mile section of highway and constituting 30 percent or more of the total number of accidents in that ½ section. The reported ½-mile section, with the ½-mile sections abutting on each end, is then scanned by ⅒-mile increments to see if there are accident clusters within short distances indicating a possible skid-prone site. Each section is then checked with the previous year's IBM run to find confirming values, if the respective sections have remained untreated with skid-resistant surface.

Check of Likely Skid-Prone Sections

For the skid-prone sections provided by the previously noted sources, a skid-trailer is used to obtain actual skid numbers. Required skid numbers are then computed for comparison. The design speed is taken at the 95 percentile of the operating speed of the respective sections as the basis for computing the skid number demand as related to geometrics, access, and sight distance. Other assumptions are that the 95 percentile deceleration rate for approaches to curves is $\frac{1}{3}$ g and for intersections is $\frac{1}{2}$ g. Requirements for high access areas is $\frac{1}{2}$ g. In addition to these demands, the demands of centrifugal force (side friction varies with the square of the speed) and of gradient of the road are taken into account. Formulas for computing the demand values (required skid numbers) are given in the Appendix. If the required skid number exceeds the expected skid number of the proposed surface treatment, consideration is given to a lower advisory speed, or a special skid-resistant treatment is recommended.

Comparison With Proposed Surface Treatment Program

The list of proposed surface treatments are compared with the list of skid-prone sites and, where duplication appears and the field review indicates no complementing corrective work is needed, the duplicated item is removed from the list of skid-prone sites. However, the required skid number is computed for the site with respect to degree of curve, grade, and 95 percentile speed, and a special mix design may be employed where the required skid number is 50 or above.

Remedial Measures and Follow-Up

A checklist of remedial measures for correction of accident-prone locations for use in a field review has been prepared. Skid-prone locations are examined to determine whether there are other contributing causes to the accidents occurring at these sites. If there are contributing causes in terms of geometrics, access, roadside parking, roadway condition, or other pertinent relationships, an analysis is made in terms of cost-effectiveness to determine whether this complementing work should be undertaken along with, or in lieu of, skid treatment.

We have developed some rule-of-thumb norms from which we can calculate expected results of corrective measures. Admittedly, results may be veiled in accident fluctuations and in other unknowns, but we believe the principle is valid regardless of the validity of the numbers of reported accidents. We believe that by appropriate corrective measures we can approach the statewide "norm" for accident rates in terms of functional systems, geometrics, ADT, and condition of roadway. For example, the statewide norm for rural level tangents in dry weather is 200; in wet weather it is 440. The statewide norm for rural winding highways is approximately 300 for dry weather and approximately 900 for wet weather. Norms have also been computed for other geometrics. When the accident rate is high in dry weather and the skid number low, we have found that an increase in skid number will reduce the dry-accident rate as well as the wet rate.

Although subjective evaluation is now required in our analysis we hope to accumulate a data bank on the relationship between changes in skid numbers and the consequent changes in skidding accidents that will reduce the judgment factor.

After making certain assumptions, we can calculate the required skid number values for curves, grades and intersections, singly or in combination (see the Appendix). The problem then is to supply that need in a suitable mix design to provide a treatment that will retain that value for 3 years or more. We can find sites where geometry dictates a need for a skid number of 60 at 40 mph. It is not easy to get and retain such a skid number. Cost-effectiveness analysis then assumes a more important function in proposing corrective measures.

Although we have available the reported property damage costs, we do not use them for the respective sites for which they are reported. We are aware that accident costs may reflect the impact of the site; however, it is felt that a better value is obtained by using the statewide average value for property damage to which we add accepted values for cost of personal injury and fatality to obtain the statewide average cost per accident. We also assume the reported 1-cent-per-vehicle-mile cost as the economic cost. This figure results in a somewhat higher cost than our calculated composite cost of about $2,000 per accident, which does not include the peripheral costs.

Special Treatment of Winding Roads on Grades

It is impracticable to vary the mix design to match skid number demands to the varying geometrics of winding roads. Instead, we provide a mix having a feasible skid number for the entire length. When that skid number will not accommodate the legal speed on a winding road, a lower advisory speed is posted, i.e., a limit that will reasonably accommodate most of the curves. Within this section, curves that are so sharp that the new skid number together with the overall advisory speed cannot adequately accommodate them are posted individually with a still lower advisory speed.

Incremental Cost-Effectiveness Appraisal

Various corrective measures can be considered—various increments of correction in geometrics, in traffic regulation and control, in roadway condition improvement, in signs and markings, in illumination and delineation, and in variant types of skid-resistant treatment. The cost of each increment of remedial measure can then be estimated.

The relation of pavement and shoulder width, of curvature, of traffic density, of weather, and of other independent variables and traffic accidents in dry and wet weather has been calculated. These are gross values, but they are useful as a guide in estimating possible changes in traffic accidents as the independent variables are changed.

With this kind of information, an incremental benefit-cost ratio can be computed for each skid-prone site. It is well to reiterate that skid-proneness is one facet linked with a multifaceted accident-proneness and that the same conditions contributing to skid-proneness may contribute to other types of accidents. It is well to remember that the correction of the condition leading to a skid might also correct a condition leading to other types of accidents.

Rank Order for Programming and Priority

We rank sites initially by order of benefit-cost ratio. Other considerations are involved in the final ranking. Public relations at times is a force in ranking.

A 6-mile section with 25 wet-pavement accidents may be judged as having a higher priority than a 1-mile section with five such accidents. The accident rate for 100 million vehicle-miles has not been too realistic in the short sections with accidents clustering at a point.

When the rank order is reviewed and adjustments are completed, we select enough projects for a feasible program. We have not established floors for skid numbers, but we can formulate a sizable de-slicking program.

PROOF TESTS AND FEEDBACK

Our field testing has been limited to 2 years of experience covering about 15 sites. On some 13 of these sites, totaling 5.73 miles in length, there was in the 12-month period before treatment a total of 101 accidents; 53 of these were on wet pavements. During the 12-month period after treatment, the 101 accidents were reduced to 53 accidents (a reduction of 48 percent) and the 53 wet accidents were reduced to 19 (a reduction of 64 percent), which is comparable to the statewide average for wet-pavement accidents. (These sections were located on US-460 between Bluefield, West Virginia, and the Virginia state line via Princeton.)

Another site of 0.62 mile had 22 accidents including 7 wet-pavement accidents. It was treated in 1969, and there was a consequent reduction to 9 total accidents (a 60 percent reduction) and to 2 wet-pavement accidents (a 70 percent reduction). This site is at Easton Hill on US-119 just east of Morgantown, West Virginia.

The terror and tragedy of spin-outs were eliminated for 66 people in 1 year in the treatment of 6.33 miles of pavement. The cost was $60,000, but twice that was saved.

ACKNOWLEDGMENT

The opinions expressed in this paper are those of the authors and not necessarily those of the West Virginia Department of Highways.

APPENDIX

FORMULAS FOR CALCULATING REQUIRED SKID NUMBER

The following symbols are used in the formulas:

- a = acceleration (or deceleration) in ft/sec^2;
- f = coefficient of friction;
- sf = side friction;
- g = gravity (32.17 ft/sec^2);
- SN = skid number = 100 ft;
- G = percentage of grade;
- V = speed in mph;
- v_1 = initial observed approach speed in ft/sec;
- v_2 = safe speed at PC or intersection in ft/sec. (This will be zero for complete stop and advisory speed for curves.);
- d = distance between point of beginning deceleration and point of completed deceleration or brake-stopping distance;
- e = superelevation rate;
- D = degree of curve; and
- r = radius in feet.

Superelevation, Side Friction, and Degree of Curve

$$e + sf = v^2/gr \qquad (1)$$

$$e + sf = [0.06689\ V^2/(5{,}729.6/D)] = (D \times 0.6689\ V^2)/5{,}729.6 \qquad (2)$$

$$sf + e = 0.0000\ 117\ DV^2 \qquad (3)$$

or

$$D = (sf + e)/(0.0000\ 117\ V^2) \approx [86{,}000\ (sf + e)]/V^2 \qquad (4)$$

For safety against centrifugal force, sf capability is assumed to be approximately f/3. Hence, if an sf of 0.12 is required, the longitudinal f would be 0.36 and the skid number would be 36.

The values given in Table 1 were calculated by using Eq. 4. Relations of skid numbers and degrees of curve are shown in Figure 1.

Grade

For each percentage point of grade, a percentage point is required in skid number. A plus grade would allow a subtraction on a minus grade; however, where passing maneuvers require both lanes, an addition is required for a plus as well as for a minus grade. The formula for 2-lane highways is

$$\text{Adjusted SN} = \text{SN} + G \qquad (5)$$

Deceleration and Brake-Stopping Distance

Brake-stopping distances for various speeds and skid numbers are given in Table 2. Relations, braking distance, and speed on curves are shown in Figure 2.

$$d = (v_2^2 - v_1^2)/2a \qquad (6)$$

TABLE 1

SPEED ZONING REQUIREMENTS FOR VARYING DEGREES OF CURVATURE

Speed (mph)	Maximum Negotiable Curve (deg)[a]			
	SN 20	SN 30	SN 40	SN 50
60	4.0	4.8	5.5	6.1
50	5.7	6.9	8.0	9.0
45	7.1	8.5	12.5	14.0
40	9.0	10.8	16.0	19.0
35	11.7	14.0	22.0	26.0
30	16.0	19.1	32.0	37.0
25	23.0	27.5	50.0	57.0
20	35.9	43.0	89.0	102.0
15	63.8	76.4		

[a]Advisory for wet pavements.

TABLE 2

BRAKE STOPPING DISTANCE AS RELATED TO SKID NUMBER AND SPEED

Speed (mph)	Brake Stopping Distance (ft)		
	SN 30	SN 40	SN 50
20	44	33	27
30	100	75	60
40	178	133	107
50	278	208	167
60	400	300	240
70	544	408	326

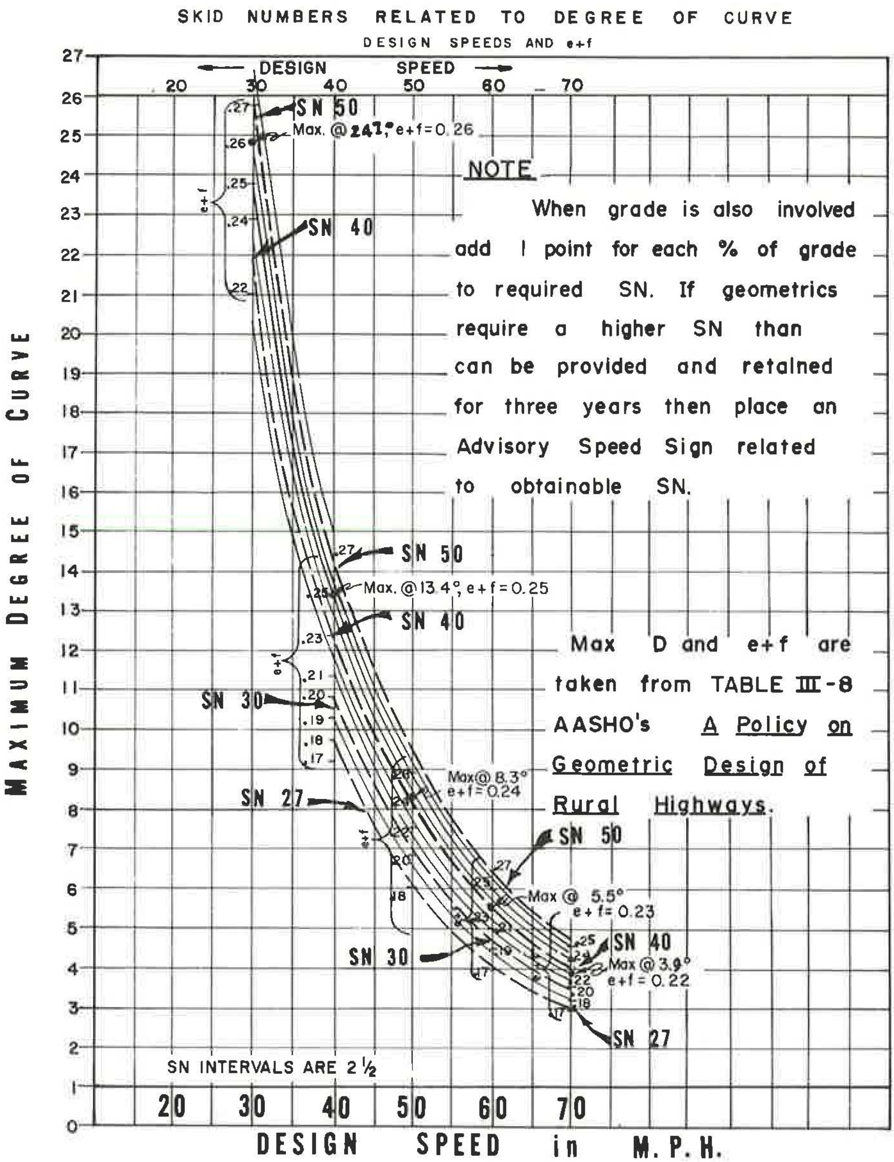

Figure 1. Skid numbers related to degree of curve.

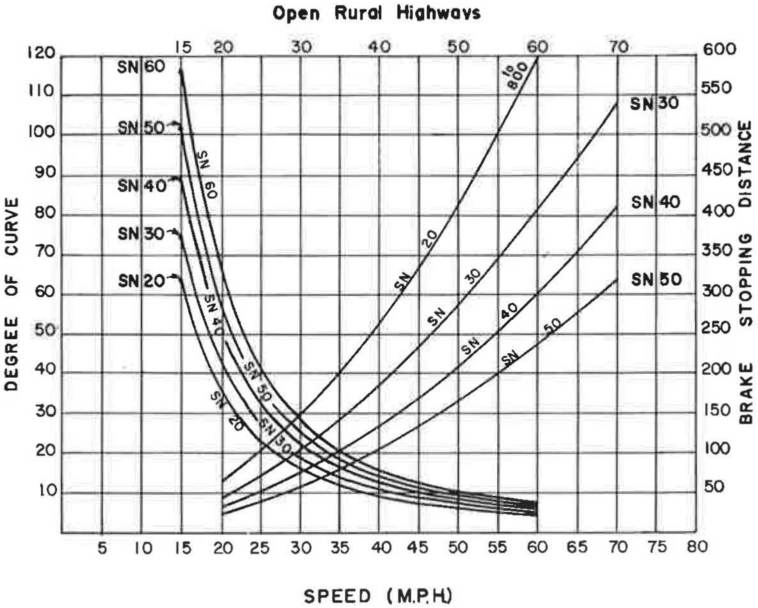

Figure 2. Skid numbers related to braking distance and safe speed on curves on open rural highways.

When the vehicle comes to a stop, we have

$$d = V^2/30f = V^2/0.30\ SN \tag{7}$$

$$d = 3.33\ V^2/SN \tag{8}$$

Because $v^2 = 2.152\ V^2$,

$$d = 2.152\ V^2/2a, \tag{9}$$

or

$$d = 1.08\ V^2/a$$

or

$$d = (1.08\ v^2)/[32.17\ (f - g)] \tag{10}$$

Hence, from Eqs. 6 and 8

$$v^2/2a = 3.33\ V^2/SN \tag{11}$$

$$v^2 = (2a \times 3.33\ V^2)/SN \tag{12}$$

$$v^2/V^2 = 6.66\ a/SN \tag{13}$$

Because $v = 1.467\ V$ and $v^2 = 2.152\ V^2$,

$$2.15\ V^2/V^2 = 2.152 = 6.667a/SN \tag{14}$$

$$SN \approx 3.11a$$

$$f = a/g \tag{15}$$

$$100\ f = SN$$

or

$$100\ f = 100a/32.17 = 3.11a \quad (16)$$

In connection with deceleration rates, the following values have been proposed:

Rate = ft/sec^2	f	Comment
10	0.31	Maximum comfortable deceleration
11.26	0.35	Minimum values
11.90	0.37	proposed by various
12.86	0.40	jurisdictions
16.08	0.50	Emergency stop

(The 95 percentile values at skid-prone site should be a determinant of f.)

SKID NUMBER DEMANDS AT SIGNALIZED INTERSECTION

Time and Distance

A vehicle can be at any distance from an intersection when the amber light appears. The driver must use his judgment as to whether he stops or proceeds through the intersection. The prudent and experienced driver can usually make the decision immediately. There are critical distances for stopping and for continuing, and they depend on vehicle speed, skid number of approach surface, and length of amber interval. The following formulas provide useful tools for calculating the critical distance and time.

$$a = v/t \quad (17)$$

$$d = 1.08\ V^2/a \quad (18)$$

$$t' = s/v;\ s = t'v \quad (19)$$

$$t = 2d/v \quad (20)$$

$$SN = 3.11a \quad (21)$$

where

a = deceleration rate;
d = brake-stopping distance;
t = amber interval less 0.5 sec for brake reaction, allowable stopping time;
t′ = travel-through time;
v = initial speed in ft/sec;
V = initial speed in mph; and
s = distance to intersection plus cross street width and vehicle length.

(It is assumed that the driver will be watching the upcoming traffic signal and be alert to its changing intervals.)

Equations 17 through 21 state that it takes almost twice as long to stop as to travel through the intersection. They say that the amber interval determines the minimum stopping distance and the maximum travel-through distance, as related to various speeds and skid numbers.

The derived data given in Table 3 and shown in Figure 3 give evidence of the need for a skid number of 50 or more at intersection approaches. The approach stopping distance, d, is the distance to the intersection. The travel-through distance, s, includes the distance through the intersection, w, and the length of the vehicle, l. s becomes d

TABLE 3

SKID NUMBERS BASED ON TIME AND DISTANCE AT SIGNALIZED INTERSECTIONS

Approach Speed (mph) (1)	Amber Interval (sec) (2)	Approach Distance on Amber: Go, d Range (3)	Go or Stop, d to s (4)	Stop, d Range (5)	Skid Number Based on Col. 4 Data (6)
20	3	0 to 37	37 to 74	73 and up	36 to 18
30	3	0 to 55	55 to 110	110 and up	55 to 27
40	3	0 to 92	92 to 183	183 and up	72 to 36
40	4	0 to 103	103 to 205	205 and up	52 to 26
50	5	0 to 165	165 to 330	330 and up	50 to 25
55	5	0 to 181	181 to 363	363 and up	56 to 28
60	6	0 to 198	198 to 396	396 and up	50 to 25

Note: No increase in initial approach speed is assumed.

at s distance from the intersection. If vehicles ahead of the subject car stop, the stopping distance for the subject car is decreased and the skid number is increased. Therefore,

$$SN = 4.56\ V/t$$

or

$$SN = 3.36\ V^2/d$$

where

t = amber interval less 0.5, and
d = s.

Although there is substantial range for judgment as to stopping and continuing through the intersection due to variances in driver judgment, there is good evidence that skid numbers should be as high as economically feasible. Although decelerations above 14 ft/sec^2 (with a skid number requirement of 44) are infrequent, it is entirely possible to have decelerations of 16 ft/sec^2 (requiring a skid number of 50) or even greater

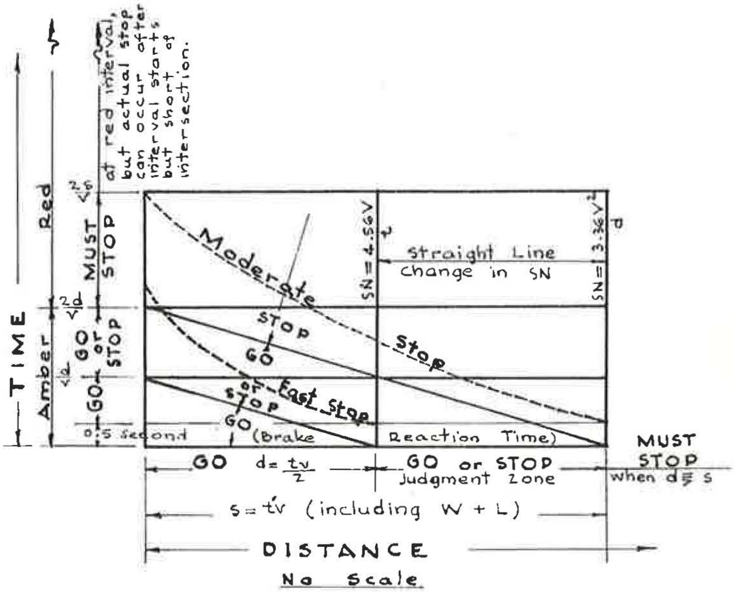

Figure 3. Time and distance relationships at signalized intersections.

frequently enough to warrant consideration. If the signalized intersection is on a grade or curve, the additional demands should be taken into account. Skid treatment, speed zoning, and increased amber interval are remedial measures.

Spot speed studies in West Virginia show the following relationships at signalized intersections with respect to speeds through green signal light in a 50-mph speed zone where the green light is followed by a 4-sec amber light.

Percentile	Speed (mph)
100	60
95	54
90	52
85	50

These figures, to say the least, tell us that even the 85 percentile vehicle can occasionally encounter serious difficulty, even on dry pavement. A comparable range in speed frequency distribution was found where a 3-second amber followed the green.

Cost-Effective Solutions at Intersections

If an appropriate skid-resistant treatment for intersections costs $20 per ton in place for a 40 lb treatment for a length of 500 ft of full width on each approach leg, then for a 24-ft roadway the cost would be approximately $2,000 per intersection.

The cost of one accident is estimated to be about $2,000. Hence, if one wet-pavement accident is prevented, the costs and benefits would balance from the standpoint of economy. In line with our expressed philosophy, we have arbitrarily set a benefit-cost ratio of 3 as the cutoff point.

PART 2
RELATED PAPERS

FRICTION AND THE MECHANICS OF SKIDDING AUTOMOBILES

Raymond M. Brach, Department of Aerospace and Mechanical Engineering, University of Notre Dame

•IT has been evident for many years that the skid resistance of automobile tires on pavement, particularly wet pavement, varies significantly with speed. Many researchers have published experimental evidence that shows that the coefficient of friction decreases with increasing speed (1, 2, 3, 4). In most cases this variation is approximately linear; in some cases it is quadratic. (Excellent, comprehensive review articles (5, 12) discuss various methods of measuring the coefficient of friction.) Despite the knowledge that friction varies with speed many people use a constant, average value for simplicity (6, 7, 8). Unfortunately, many people do not understand the nature of this simplification. As a result, some confusion and unnecessary testing have appeared in the literature. It is the intent of the following work to clear up some of the possible confusion and to illustrate that a more complete model of varying friction is possible without undue complication. Specifically it is shown that the differential equation of motion of a skidding automobile can still be integrated even when variable friction is included. An "exact" algebraic expression for the skidding distance is obtained in terms of the initial speed, weight distribution, and friction characteristics of the vehicle.

This exact expression is examined from three points of view. First, it is used to explain how the improper use of a constant, average friction value can lead to biased results. Second, it is shown that actual, variable friction curves can be found by using curve-fitting techniques with rather simple experimental data. Third, the exact expression is used from the point of view of accident investigation to show that the initial speed of a skidding automobile under very general vehicle-tire-road conditions can be read from a single graph.

DERIVATION OF EQUATIONS

Figure 1 shows a free body diagram of a vehicle, with wheels locked, skidding up a positive grade of angle θ. As shown in the figure, W is the total vehicle weight, N_1 is the total force between both rear tires and the pavement, N_2 is the total force between both front tires and the pavement, f_1 and f_2 are the total rear and front frictional forces respectively, D is the aerodynamic drag force, and ℓ_1 and ℓ_2 are the distances between the wheels and the center of gravity of the vehicle. From Newton's Law for assumed planar motion, the equation of motion in the x direction is

$$m(dv/dt) = -mg \sin \theta - f_1 - f_2 - D \tag{1}$$

where $m = W/g$, g is the acceleration due to gravity, and $v = dx/dt$ is the speed. For this type of analysis the vehicle can be treated as a rigid body; consequently, it is in equilibrium in the y direction. This gives

$$-W \cos \theta + N_1 + N_2 = 0 \tag{2}$$

Sponsored by Committee on Surface Properties-Vehicle Interaction.

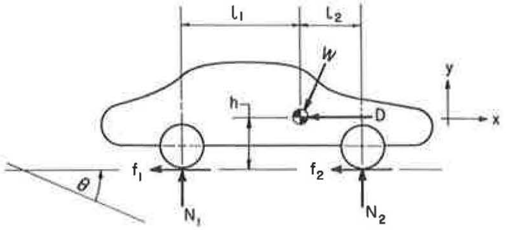

Figure 1. Free body diagram of a skidding vehicle.

Similarly because the vehicle is not rotating,

$$N_2(\ell_1 + \ell_2) - W \cos\theta\, \ell_1 + (W \sin\theta + D)\, h = 0 \qquad (3)$$

The following assumptions are made:

1. The coefficients of friction are velocity dependent such that $f_1 = N_1\mu_1(v)$ and $f_2 = N_2\mu_2(v)$, where $\mu_1 = \mu_{10} - k_1 v - h_1 v^2$ and $\mu_2 = \mu_{20} - k_2 v - h_2 v^2$.
2. The center of gravity is low, i.e., $(W \cos\theta)\,\beta >> (W \sin\theta + D)\,\gamma \approx 0$ and $(W \cos\theta)\,\alpha >> (W \sin\theta + D)\,\gamma \approx 0$, where $\alpha = \ell_1/(\ell_1+\ell_2)$, $\beta = \ell_2/(\ell_1+\ell_2)$, and $\gamma = h/(\ell_1+\ell_2)$.
3. The aerodynamic drag is proportional to the square of the velocity, i.e., $D = c_o v^2 = cWv^2$.

These assumptions can be used to reduce Eqs. 1, 2, and 3 to

$$dv/dt = -g \sin\theta - g \cos\theta\, [\beta\mu_1(v) + \alpha\mu_2(v)] - gD^* \qquad (4)$$

where $D^* = D/W$, the drag force per unit weight. Eq. 4 is the equation of motion of a vehicle skidding in a straight line. Substitution of new variables simplifies Eq. 4. That is,

$$-(1/g)\,(dv/dt) = A\,v^2 + Bv + C \qquad (5)$$

where

$A = c - (h_1\beta + h_2\alpha)\cos\theta$;
$B = -(k_1\beta + k_2\alpha)\cos\theta$; and
$C = \sin\theta + (\mu_{10}\beta + \mu_{20}\alpha)\cos\theta$.

Eq. 5 can be rearranged to a form convenient for integration.

$$\int_{v_o}^{v_f} v\,dv/(Av^2 + Bv + C) = -\int_0^d g\,dx \qquad (6)$$

For $v_f = 0$ this gives

$$\begin{aligned} -gd = {} & (1/2A)\,\ell n C - \left\{B/[2A(B^2 - 4AC)^{1/2}]\right\} \\ & \ell n \left\{[B - (B^2 - 4AC)^{1/2}]/[B + (B^2 - 4AC)^{1/2}]\right\} \\ & - (1/2A)\,\ell n\,(Av_o^2 + Bv_o + C) + \left\{B/[2A(B^2 - 4AC)^{1/2}]\right\} \\ & \ell n \left\{[2Av_o + B - (B^2 - 4AC)^{1/2}]/[2Av_o + B + (B^2 - 4AC)^{1/2}]\right\} \end{aligned} \qquad (7)$$

Here v_o is the velocity at the initiation of skidding, v_f is the final velocity, and d is the distance over which the car comes to rest. If the friction characteristics are known as well as the vehicle weight distribution, the aerodynamic drag, the grade, and the initial speed, Eq. 7 will give the skid distance for a complete stop. Some special cases of interest are when the coefficient of friction depends linearly on the velocity ($h_1 = h_2 = 0$) and when the coefficient of friction is a constant ($h_1 = h_2 = k_1 = k_2 = 0$). These cases are respectively as follows:

1. A = 0 (aerodynamic drag also neglected).

$$\ell n[(B/C)\,v_o + 1] - (B/C)\,v_o = -(gdB^2/C) \qquad (8)$$

2. A = B = 0 (aerodynamic drag also neglected).

$$v_o = (2Cgd)^{1/2} \tag{9}$$

For level ground, $\theta = 0$. When the front and rear vehicle weights are equal, Eq. 9 further reduces to the "standard" stopping distance formula

$$d = v_o^2/2fg \tag{9a}$$

where f is an "average" coefficient of friction, which is called a "friction factor" in the sequel to distinguish it from the coefficients of friction, μ_1 and μ_2.

USE OF THE STOPPING-DISTANCE FORMULA

Two common methods of measuring the friction properties of tires on particular pavements are the stopping-distance method and the skid-trailer method. In the former, the vehicle is brought up to a given speed, the brakes are locked, and the vehicle skids to a stop. The skid distance is measured, and Eq. 9a is used to calculate a friction factor. In the latter method (12), a special trailer is pulled over a pavement at a constant speed with the wheels locked. The wheel torque can be measured, and the coefficient of friction can be calculated. Both methods are generally used for various speeds, and friction-speed curves can be plotted.

One important distinction between these two methods is made here. The skid trailer measures an "instantaneous" value of friction for a given speed, whereas the stopping-distance method yields an "average" value over a range of speeds, namely the friction factor.

Figure 2 shows a hypothetical friction coefficient that decreases quadratically from 0 to 80 mph, namely,

$$\mu = 0.6 - (3.25 \times 10^{-3})v + (1.11 \times 10^{-5})v^2$$

For zero grade and no aerodynamic drag, Eq. 7 yields the skidding distance shown in Figure 3. If one takes corresponding values of initial speed and stopping distance shown in Figure 3 (which is what one gets from stopping-distance experiments) and computes various values of the friction factor, f, from Eq. 9a, the friction factor curve shown in Figure 2 results. Any friction value from this curve can be used with Eq. 9a to compute stopping distances. On the other hand, skid-trailer data are instantaneous coefficient

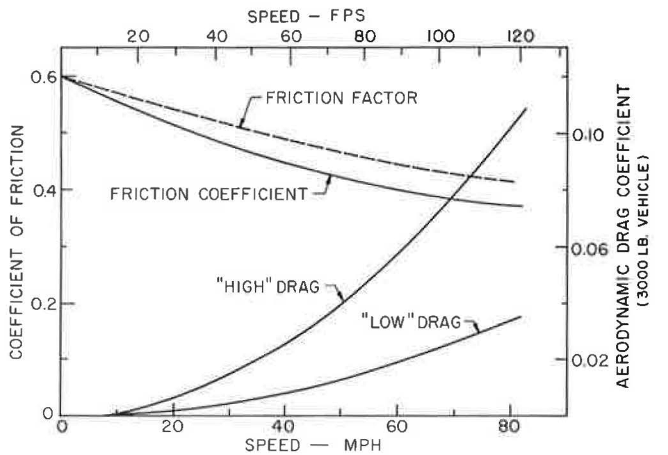

Figure 2. Friction and aerodynamic drag.

of friction values such as the friction coefficient curve shown in Figure 2. If these values of μ are used with Eq. 9a, a larger stopping distance is calculated than is obtained in practice because the friction coefficient is smaller than the friction factor. This should explain, at least in part, why 90 percent of 3,900 stopping-distance measurements (8) were smaller than the calculated stopping distances.

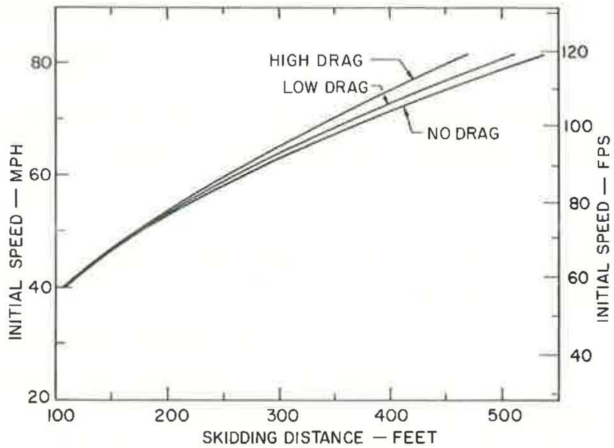

Figure 3. Skidding distance computed by using Eq. 7.

MEASUREMENT OF COEFFICIENTS OF FRICTION

This section discusses two topics: the importance of aerodynamic drag and how to obtain the instantaneous friction coefficient curve from stopping distance measurements.

For most practical purposes, the drag force is proportional to the square of the speed, i.e., $D = c_o v^2$. It depends essentially on the size and shape of the vehicle and generally ranges from 50 to 250 lb at a speed of 60 mph (9, 10). Quite obviously, in a low friction situation (say, on glare ice) a high drag force can be significant, particularly at high speeds. However, in typical test conditions (at speeds below 60 mph) drag is usually negligible. This is illustrated in Eq. 7 by solving for two values of drag: one designated as a "high" value, $c_o = 7.4 \times 10^{-3}$, and another designated as a "low" value, $c_o = 2.2 \times 10^{-2}$, also shown in Figure 2. (These are within the stated range.) The "typical" friction curve shown in Figure 2 is assumed. The results are shown in Figure 3 along with the zero drag case, $c_o = 0$. At speeds of less than 60 mph the percentage error in stopping distance with drag neglected is 8 percent or less. If this size of error is not tolerable or if the friction curve is considerably lower than the one shown in Figure 2, drag must be accounted for. Drag is neglected, however, in the remainder of this paper in order to simplify the results.

Generally, for safety reasons, tests to find the instantaneous friction curve from stopping-distance data involve speeds of less than 60 mph. Because in this range a linear relationship between friction and speed is usually found, and also to simplify the presentation, Eq. 8 is used. (Eq. 7 could be used with little additional difficulty.) Further, testing is generally done under controlled conditions, e.g., on level terrain ($\theta = 0$) and with identical tires, both front and rear. Thus $C = \mu_o$, the friction curve intercept, and $B = -k$, the slope of the friction curve. Equation 8 can then be rewritten as

$$d_i = -(10/a) \ln (1 - bv_i) - (10/a)\, bv_i \qquad (10)$$

where $a = 10k^2g/\mu_o$, $b = k/\mu_o$, and the subscript i indicates different experimental values of stopping distance for various initial speeds v. From the viewpoint of curve-fitting, Eq. 10 has two unknowns (a and b) appearing in nonlinear form. By using the classical method of differential corrections (11), now called a Newton-Raphson technique, we can minimize the sum of squares of deviations of d_i from the true stopping distance d.

$$Q = \sum_{i=1}^{n} (d_i - d)^2$$

with respect to a and b for all n experimental values. This furnishes a set of equations solvable for a and b.

Experimental data are required to illustrate the applicability of this technique. Because none was available to the author, some computer "experiments" were performed. First, a coefficient of friction curve was chosen (Fig. 4). For speeds of 20, 30, 40, 50, and 60 mph, two values of stopping distance were calculated from Eq. 8, each with a different, normally distributed "error" added. This was done by using a random number generating subroutine on an IBM 1130 digital computer. These fictitious experimental values are given in Table 1. The values of a and b were estimated by using the curve-fitting technique cited perviously. All numerical values are given in Table 1, and all results are shown in Figure 4. The exact stopping-distance curve from Eq. 8 and the curve fit from experimental data are essentially identical. The friction curves, exact and experimental, although not identical are very close. The maximum error in estimating μ (v) is 5.9 percent at v = 0. This example illustrates the usefulness and practicality of finding the friction curve by using nonlinear curve-fitting methods and data from stopping-distance experiments. One point of caution must be mentioned. When Eq. 8 is fitted for a and b, if the data indicate that the value of k is near zero, the natural logarithm in Eq. 8 must be expanded in a series to avoid differences of large numbers. This can be done automatically in the computer solution and presents no particular problem.

ACCIDENT STUDIES

A common situation occurs in accident investigation where a vehicle leaves measurable skid marks from a panic stop. When the car skids to a complete stop or when the speed at the end of the skid is known (or can be estimated), Eq. 7 can be used to furnish an expression for the initial speed v_o. For simplicity, it is assumed that the velocity at the end of the skid, v_f, is zero, although this is not necessary. For convenience, Eq. 8 will be used in the following examples.

In this situation, where everything is presumed known except v_o, Eq. 8 is best solved either by a computer using a root-finding method or by solving the equation for various initial velocities until the known stopping distance is found. In both cases, the generality and convenience of Eq. 8 is remarkable. Specifically, Eq. 8 takes into account the following:

1. Grade angle, θ;

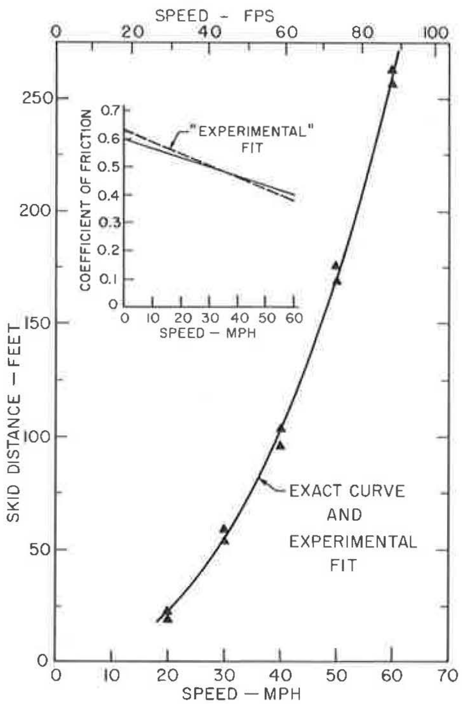

Figure 4. Least-squares curves from stopping-distance experiments.

TABLE 1

STOPPING-DISTANCE DATA

Actual Values, Eq. 8[a]		Values With Random Error[b]		Values From Fitted Curve[c]	
v (ft/sec)	d (ft)	v (ft/sec)	d (ft)	v (ft/sec)	d (ft)
29.3	24.1	29.3	19.8	29.3	23.1
		29.3	22.9		
44.0	56.5	44.0	59.3	44.0	54.7
		44.0	54.3		
58.7	105.1	58.7	103.7	58.7	102.6
		58.7	95.5		
73.3	172.1	73.3	175.2	73.3	170.0
		73.3	168.9		
88.0	260.5	88.0	257.0	88.0	260.7
		88.0	262.6		

[a] μ_o = 0.600 and k = 2.27 x 10^{-3}.
[b] μ_o = 0.600 and k = 2.27 x 10^{-3}. Error in the stopping distance is normally distributed with a mean of zero and a standard deviation of 5 ft.
[c] μ_o = 0.635 and k = 2.83 x 10^{-3}.

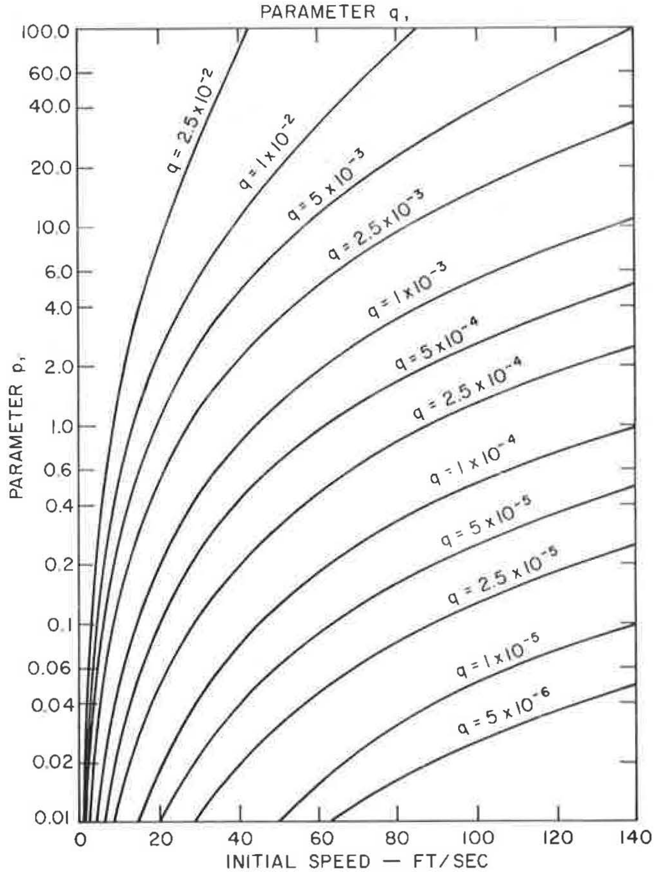

Figure 5. Initial speed from stopping distance.

2. Nonuniform weight distribution of the vehicle, α and β; and
3. Variable friction, μ_o and k, which can differ from the front to the rear of the vehicle.

Furthermore, all of these data are combined into only two parameters, $q = -B/C$ and $p = -gdB$. In other words, once all of the friction data, weight distribution data, grade, and stopping distance are known, the initial speed depends only on p and q. Consequently, all of the information from Eq. 8 can be represented by a family of curves that gives the initial velocity v_o as a function of p and q. Figure 5 shows these curves for most frequently encountered values of p and q.

As an example of the use of the curves shown in Figure 5, suppose the following is known:

1. $\theta = 0.087$ radians (5 deg upgrade);
2. $\alpha = 0.55$ and $\beta = 0.45$ (55 percent of weight on front tires);
3. $\mu_{o1} = 0.6$, $\mu_{o2} = 0.5$, $k_1 = 0$, and $k_2 = 2 \times 10^{-3}$ (rear tires more effective than front tires); and
4. Stopping distance, $d = 100$ ft.

For these values, $q = -B/C = 1.9 \times 10^{-3}$ and $p = -gdB = 3.523$. From Figure 5, this gives an approximate initial speed of 62 ft/sec or 42 mph.

CONCLUSIONS

The major point demonstrated in this paper is that a rather general mathematical model of a skidding automobile can be constructed and solved with little difficulty. The solution, or simpler forms of the solution, can be useful in curve-fitting of experimental data from stopping-distance experiments. Further, the solution can also be used to obtain initial speeds in the case where the skid distance and friction characteristics are known. Finally, it was shown that the actual, instantaneous values of coefficient of friction should not be used with the standard stopping-distance formula, Eqs. 9 or 9a, but only with the more exact form, Eqs. 7 or 8. Conversely, "average" friction data should not be used with Eqs. 7 or 8, but only with Eqs. 9 or 9a.

NOTATION

The following symbols were used in the equations in this paper.

A = definition given following Eq. 5;
a = unknown coefficient (Eq. 10);
B = definition given following Eq. 5;
b = unknown coefficient (Eq. 10);
C = definition given following Eq. 5;
c = aerodynamic drag coefficient, lb-sec^2/ft^2;
c_o = aerodynamic drag coefficient, sec^2/ft^2;
D = aerodynamic drag force;
d = distance of skid;
f = friction factor (average coefficient over a variable speed skid);
f_1, f_2 = friction force between tires and pavement;
g = acceleration due to gravity;
h = perpendicular distance from pavement to vehicle, cg;
h_1, h_2 = coefficients in friction in expression;
k_1, k_2 = coefficients in friction in expression;
ℓ_1, ℓ_2 = length of vehicle;
m = mass of vehicle;
N_1, N_2 = normal force between tires and pavement;
p, q = parameters defined in preceding section;
v = velocity of vehicle, ft/sec;
v_f = velocity of vehicle at end of skid, ft/sec;
v_o = velocity of vehicle at initiation of skid, ft/sec;
W = total vehicle weight;
x = position coordinate of vehicle;
α = $\ell_1/(\ell_1 + \ell_2)$;
β = $\ell_2/(\ell_1 + \ell_2)$;
γ = $h/(\ell_1 + \ell_2)$;
θ = angle of grade (positive for upward motion); and
μ_1, μ_2 = coefficients of friction.

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COMPARISON OF HIGHWAY PAVEMENT FRICTION MEASUREMENTS TAKEN IN THE CORNERING-SLIP AND SKID MODES

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Friction tests using smooth and treaded tires with 10- and 24-psi tire-inflation pressures on wet and dry surfaces were taken with a Mu-meter and the Texas Highway Department research skid trailer. Fifteen pavement surfaces that exhibited widely different friction levels, friction-velocity gradients, drainage capabilities, mineralogical properties, and texture classifications were investigated. Pavement macrotexture tests were conducted by volumetric and mechanical roughness detector methods. Comparisons and relationships between various friction parameters as obtained with both instruments were made. Statistical analyses and typical plots are given. Friction tests obtained with both instruments compared favorably, provided similar tire tread configurations were used. On an average, slightly higher friction forces were available in the slip mode of operation (measured by Mu-meter) than in the skid mode (measured by skid trailer). The importance of providing adequate drainage in the tire-pavement contact area is stressed. Tests made with smooth and treaded tires in both the slip and skid mode emphasized the importance of pavement surface macrotexture at speeds of 40 mph or more.

•FRICTION measurements of tire-pavement interaction are considered highly acceptable for evaluating the skid-resistant properties of pavement surfaces and are essential to the determination of what occurs at the tire-pavement interface under different environmental conditions. Research by numerous investigators has shown that experimental studies under actual field conditions are a necessary supplement to theoretical analyses and laboratory investigations. For this reason, the work reported in this study was field-oriented.

Skid resistance is often reported as a friction coefficient, or as the ratio of the friction force (drag) to the load of the bodies sliding over each other. More recent practice is to multiply the friction coefficient by 100, report the value as a whole number, and call it a skid number. A skid number is valid for specific conditions only, that is, for the tester and pavement combination and the environmental operational conditions present. Similar reasoning may be applied to the cornering-slip mode.

Attempts have been made to characterize the skid-resistant properties of pavement surfaces in a qualitative manner such as surface macrotexture, drainage characteristics of the road surface, and aggregate size, shape, microtexture, and mineralogy. The majority of these are not convenient survey measures nor has the relative magnitude of their influences been universally accepted; thus, characterizations at present are mainly dependent on implicit information from friction tests.

The principal causes of pavement slipperiness are (a) the presence in the tire-pavement contact area of water that, with increasing vehicle speeds, lowers the obtainable frictional drag and raises the frictional demand and (b) higher traffic volumes

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that, through pavement wear and aggregate polish, drastically reduce built-in friction potential of most new pavement surface types.

Many parameters affect the interactions at the tire-pavement interface. The principal ones are (a) mode of operation, (b) pavement surface characteristics, mainly macroscopic and microscopic roughness and drainage capability, (c) water-film thickness at the interface, (d) tire-tread depth and elastic and damping properties of the tire rubber, and (e) vehicle speed. Thus, if friction coefficients are to be meaningful for evaluation or comparison purposes, the foregoing factors must be given consideration. Standardization of certain friction testing procedures and equipment can naturally reduce the number of variables used in survey work. Ideally, pavement surface type would remain as the only variable and, for the test mode used, differences in friction values could be attributed to this factor.

The American Society for Testing and Materials Committee E-17 has contributed greatly to the standardization of friction testing methods. One tentative and one standard method have been sponsored and approved by ASTM Committee E-17 and accepted by the Society—ASTM Designation E 274-65T (Skid Resistance of Pavements Using a Two-Wheel Trailer) and ASTM Designation E 303-66 (Measuring Surface Frictional Properties Using the British Portable Tester) respectively. In addition, ASTM Designation E 249-66 (Standard Tire for Pavement Tests) has been adopted as a standard.

For research purposes it is desirable to use more than one type of measuring mechanism. This provides information concerning the relative slipperiness of given pavement surface types under different modes and, in addition, with judicious use of other factors, friction properties of certain pavement surface types can be better evaluated under different operating conditions.

Experiments have shown that different friction levels must be expected for variable, but normal, operating modes of a tire, i. e., rolling and slipping during braking, driving, and cornering (1, 2, 3). Skidding is not a normal operating mode because the vehicle is essentially out of control when this condition exists. It has been determined from theory and experiments that the friction developed between a pavement surface and a tire operating under slip depends, for the most part, on the quantity of slip and that maximum friction occurs at about 10 to 20 percent slip (4). Primarily slip resistance has been found to reflect the adhesion properties and skid resistance, the hysteresis properties of a given tire-pavement matching (4). The question as to whether skid or slip is the better mode for evaluating potential slipperiness of pavement surfaces has been discussed by Meyer and others (4, 5). They have stated the following:

> It is arguable that skid resistance is more significant from the safety standpoint than slip resistance, on the grounds that it is most important that a vehicle come to the quickest possible stop once it is out of control. On the other hand, one can take the stand that the critical slip resistance is more important because it defines the point up to which the vehicle will remain under control.

It might also be added that the most effective braking occurs during the slip mode. In total lockup, frictional drag is significantly reduced compared to the drag for the 15 to 20 percent slip mode.

The purpose of this paper, however, is not to question which mode is the better or to discuss the mechanics and mechanisms of the two modes but, rather, to present data obtained with both modes and on various types of surfaces under stated conditions. Data comparisons are given with due regard for test variables. Properties of the pavement surfaces that are reflected in the test results are discussed.

CLASSIFICATION OF SURFACES TESTED

Previous research has indicated that pavement surfaces of a given type, i. e., asphalt concrete, portland cement concrete, and surface treatments, vary tremendously in skid-resistant properties. This variation is primarily a function of the type of aggregate contained in the particular surface. It is conceivable that aggregate type affects, to a similar degree, cornering-slip-resistant properties. Thus, it was decided that, to adequately investigate and compare cornering-slip- and skid-resistant characteristics, measurements would have to be made and analyzed for several types of pavement surfaces.

The term surface as used in this paper is defined as a section of pavement on which the wearing course is essentially identical over the entire length under study. Fifteen pavement surfaces were tested: (a) 9 hot-mix asphalt concretes, (b) 2 portland cement concretes, (c) 3 chip-sealed surface treatments, and (d) 1 flushed seal. Surfaces were chosen so as to exhibit widely different friction levels, friction-velocity gradients, drainage capabilities, mineralogical properties, and textural classifications. The surfaces were classified as to the mineralogy, size, and shape of the coarse aggregate they contained. This information is given in Table 1.

EQUIPMENT USED FOR FRICTION TESTS

The Soiltest ML-400 Mu-meter friction recorder and the Texas Highway Department research skid trailer were used to measure cornering and skid-resistance respectively (Figs. 1 and 2).

Mu-Meter

This instrument is a continuously recording, friction-measuring trailer that determines the frictional characteristics of treadless tires operating in the cornering-slip mode (12, 17, 18, 19). It measures the cornering force generated between the test surface and the pneumatic tires on two running wheels that are set at a fixed 7½-deg toe-out (yaw) angle to the line of drag.

In operation, friction produced as the running wheels are moved forward over the surface is sensed by a load cell. The resulting hydraulic pressure is transmitted

TABLE 1

DESCRIPTION OF THE FIFTEEN SURFACES

Surface Number	Route	County	Surface Type	Aggregate Type	Aggregate Size[a]	1968 Average Daily Traffic	Construction Date
3	Texas-6	Brazos	Hot mix	Lignite boiler slag	$^3/_{16}$	4,200	1965
4	Texas-6	Robertson and Falls	Hot mix	Rounded river gravel	$^5/_8$	1,420	1968
11	Texas-14	Limestone	Hot mix	Crushed river gravel	$^1/_2$	3,655	1967
13	US-84	Freestone	Hot mix	Crushed sandstone	$^3/_8$	1,310	1965
17	Farm-1687	Brazos	Hot mix	Open-graded lightweight	$^3/_8$	700	1968
18	Farm-1687	Brazos	Hot mix	Open-graded lightweight	$^5/_8$	700	1968
22	Texas-14	Limestone	Portland cement concrete	Rounded river gravel	$1^1/_2$	920	1936
28	Farm-2038	Brazos	Surface treatment	Rounded river gravel	$^5/_8$	135	1968
31	Texas-30	Grimes	Surface treatment	Crushed limestone	$^3/_8$	820	1968
33	Farm-416	Navarro	Surface treatment	Lightweight	$^1/_2$	100	1964
T-1	Texas A&M	Brazos	Hot mix	Rounded river gravel	$^5/_8$	None	1968
T-2	Texas A&M	Brazos	Hot mix	Crushed river gravel	$^1/_4$	None	1968
T-3	Texas A&M	Brazos	Hot mix, Terrazzo finish	Crushed limestone	$^1/_2$	None	1968
T-4	Texas A&M	Brazos	Clay-filled tar emulsion (Jennite) seal			None	1968
T-5	Texas A&M	Brazos	Portland cement concrete	Rounded river gravel	$1^1/_2$	None	1953

[a]All aggregates top size.

Figure 1. Mu-meter friction trailer.

Figure 2. Texas Highway Department research skid trailer.

through a flexible line to the recorder's Bourdon tube and indicating mechanism. The recorder stylus makes a trace on the moving pressure-sensitive chart paper. A third wheel serves, in effect, as a recorder drive mechanism. Split-rim wheels are used, and the tires are pneumatic, 6-ply, size 4.00 × 16 with smooth treads. Under normal operating conditions, 10- and 30-psi tire pressures are used in the running and recording wheels respectively. Friction values are read directly from the chart paper, multiplied by 100, and reported as cornering-slip numbers at the corresponding test velocity. Gradient (or slope) of the cornering-slip number-velocity curve is then calculated (the numerical difference of the cornering-slip numbers at 20 and 60 mph divided by the velocity difference of 40 mph).

$$\text{Gradient} = (SN_{20} - SN_{60})/40$$

The percentage of decrease in friction between 20 and 60 mph, termed percentage of gradient, was calculated. This takes into account the fact that the absolute decrease in cornering-slip number above 20 mph will be influenced to some extent by the cornering-slip number at that velocity. A curve of a given gradient positioned low on the friction-velocity graph would have a higher percentage of gradient than would a curve with the same gradient positioned high on the graph. If a surface has low friction at 20 mph, the decrease at higher velocities cannot be large. Thus, percentage of gradient is defined as the percentage of the gradient (obtained under test conditions) to a theoretical gradient if the cornering slip number at 60 mph were zero.

$$\begin{aligned}\text{Percentage of gradient} &= \{[(SN_{20} - SN_{60})/40]/(SN_{20} - 0/40)\} \times 100 \\ &= [(SN_{20} - SN_{60})/SN_{20}] \times 100\end{aligned}$$

Trailer

This instrument, used by the Texas Highway Department, conforms substantially to requirements of ASTM Designation E 274-65T. It utilizes the E-17 circumferentially grooved, treaded tires inflated to 24 psi. The drag forces are measured with strain gages, and the self-watering system uses a centrifugal pump that applies approximately 0.020-in. water-film thickness to the pavement surface. The development and calibration of the trailer are given elsewhere (9, 20).

Force values were taken from the chart paper, converted to friction coefficient values, multiplied by 100, and reported as skid numbers at the corresponding test velocity. Gradient and percentage of gradient were calculated in the same manner as explained previously except appropriate skid numbers were used.

FRICTION-TESTING PROCEDURES AND CONDITIONS

Documented research indicates that the drainage capability of a given surface, as determined from skid tests, varies considerably with respect to test velocity, water-film thickness, tire-tread depth, and inflation pressure. E-17 circumferentially grooved, treaded tires inflated to 24 psi are normally used at test speeds of 40 mph on pavement surfaces with approximately 0.020 in. of water-film thickness as a basis for reporting and comparing pavement skid resistance. These standard conditions were used in an attempt to better evaluate their relative effects on the cornering-slip and skid modes. In addition, other variations were incorporated into the study to gain a better insight of the overall problem.

Two series of 20-, 40-, 60-, and 80-mph friction tests were conducted with each instrument under different conditions and at four places on each surface. On several surfaces, 80-mph tests were not attempted because of poor roadway geometrics or high traffic densities. Instead, tests at top speeds of less than 80 mph were taken on these surfaces. Reported cornering-slip and skid numbers for a given test method on each surface represent average values for four places tested on that particular surface.

The testing sequence at each place was as follows:

1. A series of 20-, 40-, 60-, and 80-mph tests with the Mu-meter on dry pavement;
2. A series of 20-, 40-, 60-, and 80-mph tests with the trailer on pavement wet by the trailer's self-contained, internal watering system; and
3. A series of 20-, 40-, 60-, and 80-mph tests with the trailer and Mu-meter on pavement wet by a separate water truck.

In the third sequence, the measurements were taken concurrently with the Mu-meter lagging approximately 100 ft behind the trailer at each respective test speed. Measurements were made in the wheelpath with the position of the Mu-meter wheels nearly the same as that of the skid trailer wheels. Comparisons made between the two devices require that careful consideration be given to this factor, particularly if those data being compared came from a highway with high traffic volumes and especially if the pavement surface shows evidence of being worn and polished in the wheelpath.

The trailer watering system was calibrated to supply sufficient water to create a surface film 0.020 in. thick on the pavement. Procedures for wetting with the water truck were planned to ensure an equivalent water-film thickness. This procedure required wetting the pavement at a controlled rate with three passes of the water truck, prior to the 20-mph test. The first two passes were applied merely to cool the pavement to effect a constant evaporation rate and to wet the pavement so that an incipient runoff condition would exist. A third pass was required to obtain the 0.020-in. water-film thickness for the 20-mph test. Prior to the second and each succeeding test at a location, i.e., before the 40-, 60-, and 80-mph tests were made, an additional watering was required to replenish water lost by evaporation, splash, and runoff.

Times between watering and testing were varied from 30 to 90 sec from one surface to another and from day to day to compensate for varying pavement cross slopes, ambient temperatures, wind velocities, and humidities. This was necessary to maintain a constant volume of water on the pavements.

Test equipment and conditions are given in Table 2. The tests were conducted during August and September 1969. Air temperatures were generally in the 80- to 95-deg range, and the rainfall had been abnormally low for approximately 60 days preceding the tests. No seasonal or temperature corrections were applied to the friction numbers.

MACROTEXTURE TESTS

Numerous methods have been employed to directly or indirectly measure pavement surface macrotexture, including the sand patch test, mechanical roughness detectors, the grease smear test, the outflow meter, impression techniques, light reflection, and stereo-photography. The two procedures used in this study, profilograph and putty impression, represent examples of a mechanical roughness (profile) detector and an impression (volumetric) technique respectively. Details of the profilograph method

TABLE 2

TEST EQUIPMENT AND CONDITIONS

Test Condition Number	Equipment	Tires		Surface Condition	Wetting System
		Pressure (psi)	Type		
TC-1	Mu-meter	10	Smooth	Dry	
TC-2	Trailer	24	E-17 circumferentially grooved	Wet	Internal
TC-3	Trailer	24	E-17 circumferentially grooved	Wet	External
TC-4	Mu-meter	10	Smooth	Wet	External
TC-5	Mu-meter	24	Smooth	Dry	
TC-6	Trailer	24	Smooth	Wet	External
TC-7	Mu-meter	24	Smooth	Wet	External

have been reported by Ashkar (8), Gallaway (10, 11), and Rose (6), and details of the putty impression method have been reported by Stephens (7), Gallaway (10, 11), and Rose (6).

An average of five tests were taken at each of the four places friction measurements had been taken previously for a total of 20 per surface (test pavement). Individual test spots at each place were located in the outer wheelpath, spaced approximately 50 ft apart.

ANALYSIS OF DATA AND DISCUSSION OF RESULTS

A tabulation of cornering-slip and skid numbers, friction-velocity gradients, and percentage of gradients are given in Table 3. Macrotexture measures are given in Table 4. Average cornering-slip and skid numbers and average gradients and percentage of gradients are also given in Table 3.

Average friction number-velocity values for the test surfaces are plotted for the seven test conditions and shown in Figure 3. Ten of the surfaces were tested under five different test conditions (Fig. 3b). Data were obtained on an additional five surfaces

TABLE 3

CORNERING-SLIP AND SKID NUMBERS, FRICTION-VELOCITY GRADIENTS, PERCENTAGE OF GRADIENTS,

Surface Number	TC-1					TC-5				TC-4					TC-7	
	SN_{20}	SN_{40}	SN_{60}	Gradient	Percentage of Gradient	SN_{20}	SN_{40}	SN_{60}	Gradient	SN_{20}	SN_{40}	SN_{60}	Gradient	Percentage of Gradient	SN_{20}	SN_{40}
3	83	82	80	0.08	4	72	71	71	0.03	46	29	19	0.68	59	52	28
4	64	64	65	0.00	0					48	33	25	0.57	48		
11	56	56	56	0.00	0					44	32	25	0.47	43		
13	70	69	69	0.03	1	71	70	71	0.00	67	50	40	0.68	40	68	52
17	76	76	75	0.03	1	81	79	77	0.10	67	68	68	0.00	0	73	72
18	77	77	76	0.03	1	79	77	78	0.03	69	71	71	0.00	0	73	73
22	73	73	73	0.00	0					54	38	30	0.60	44		
28	79	80	79	0.00	0					52	53	48	0.10	8	46	42
31	80	78	77	0.08	4					70	55	35	0.88	50		
33	62	61	59	0.08	5					61	59	56	0.13	8		
T-1	69	68	69	0.00	0	67	67	67	0.00	62	47	36	0.65	42		
T-2	72	72	71	0.03	1	69	67	67	0.05	66	62	58	0.20	12		
T-3	68	67	67	0.03	1	67	68	68	0.00	68	42	25	1.07	63		
T-4	67	67	67	0.00	0	69	67	67	0.05	39	19	10	0.73	74	44	20
T-5	73	73	71	0.05	3	76	73	73	0.08	56	38	27	0.73	52	64	50
Average for Number of																
15	71	71	70	0.03	1					58	46	38	0.50	36		
10	73	73	72	0.03	1					59	48	40	0.48	36		
5	77	77	76			76	74	74		61	54	49			62	53

Note: For the Mu-meter tests, TC-1, TC-5, TC-4, and TC-7, SN = slip number; for the trailer tests, TC-2, TC-3, and TC-6, SN = skid number.

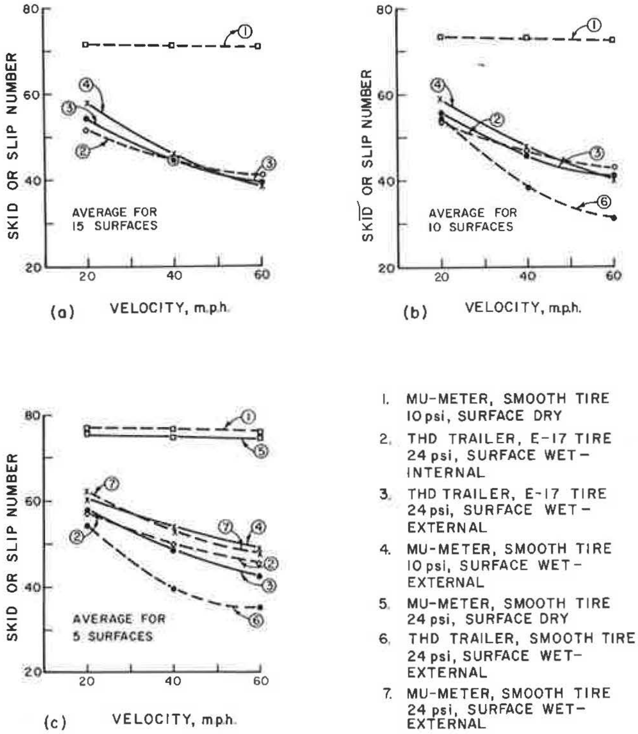

Figure 3. Average friction-velocity comparisons for different test conditions in the skid and cornering-slip modes.

AND MACROTEXTURE VALUES FOR SURFACES AND TEST CONDITIONS

TC-7		TC-2				TC-3					TC-6					Surface Number
SN_{60}	Gradient	SN_{20}	SN_{40}	SN_{60}	Gradient	SN_{20}	SN_{40}	SN_{60}	Gradient	Percentage of Gradient	SN_{20}	SN_{40}	SN_{60}	Gradient	Percentage of Gradient	
19	0.83	50	40	35	0.38	54	40	33	0.52	39	36	17	15	0.52	58	3
		37	31	29	0.20	41	30	23	0.45	44						4
		35	30	32	0.08	43	35	31	0.30	28						11
41	0.68	66	58	54	0.30	65	53	44	0.52	32	57	32	28	0.73	51	13
70	0.08	66	57	49	0.43	64	55	49	0.37	23	68	55	53	0.37	22	17
72	0.03	65	58	50	0.38	65	57	50	0.37	23	71	60	53	0.45	25	18
		50	40	35	0.38	51	40	33	0.45	35						22
39	0.18	41	36	39	0.05	42	38	39	0.08	07	39	34	30	0.23	23	28
		47	38	31	0.40	57	43	29	0.70	49						31
		64	63	62	0.05	69	62	59	0.25	15						33
		50	42	38	0.30	64	53	46	0.45	28	54	40	30	0.60	44	T-1
		54	48	44	0.25	63	55	47	0.40	25	67	51	38	0.73	43	T-2
		72	64	61	0.28	68	59	51	0.43	25	77	45	27	1.25	65	T-3
12	0.80	26	18	17	0.23	26	16	13	0.33	50	29	15	10	0.47	66	T-4
41	0.58	49	46	40	0.23	49	40	36	0.33	27	46	33	27	0.47	41	T-5
Surfaces Tested																
		52	45	42		55	45	39	0.40	30						15
		54	47	43		56	46	41	0.38	28	54	38	31	0.58	44	10
48		58	50	45		58	49	43			54	40	36			5

with only four test conditions (Fig. 3a). Figure 3c shows complete data as obtained with the seven conditions on five surfaces.

The Mu-meter results indicate that cornering-slip numbers are not affected by velocity increase on dry pavements. On wet pavements, both Mu-meter (smooth tire) and trailer (E-17 tire) results reflect the characteristic decrease in friction with increased velocity. On the average, at 20 mph, the Mu-meter indicates slightly higher friction than does the trailer; whereas at 60 mph, both instruments indicate the same magnitude (Fig. 3a, test conditions 3 and 4). Results from the trailer operating with a smooth tire (test condition 6) compared favorably with

TABLE 4

AVERAGE MACROTEXTURE MEASUREMENTS

Surface Number	Depth by Putty Impression (in.)	Peak Height by Profilograph (in.)
3	0.0090	0.0212
4	0.0234	0.0252
11	0.0340	0.0279
13	0.0182	0.0182
17	0.0224	0.0190
18	0.0412	0.0333
22	0.0115	0.0191
28	0.0563	0.0570
31	0.0432	0.0174
33	0.0648	0.0557
T-1	0.0224	0.0235
T-2	0.0235	0.0195
T-3	0.0093	0.0149
T-4	0.0019	0.0136
T-5	0.0280	0.0203

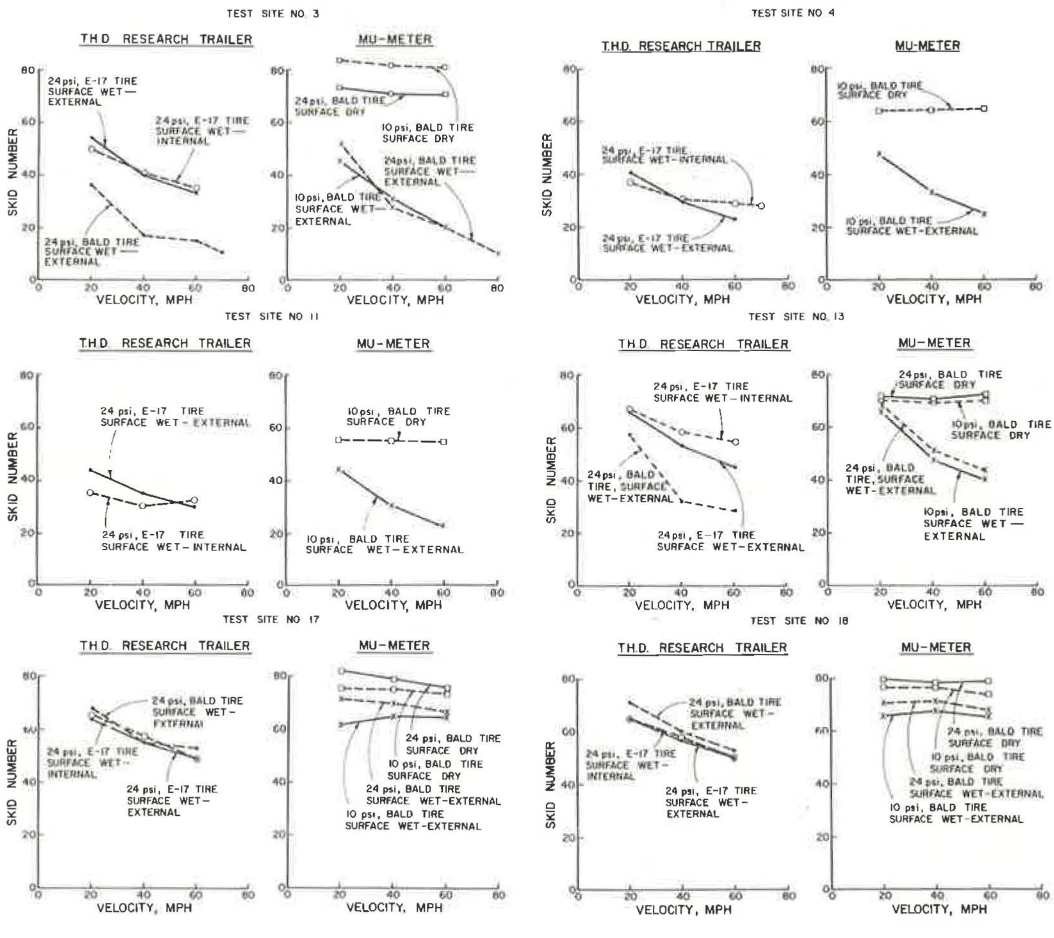

Figure 4. Trailer and Mu-meter friction values.

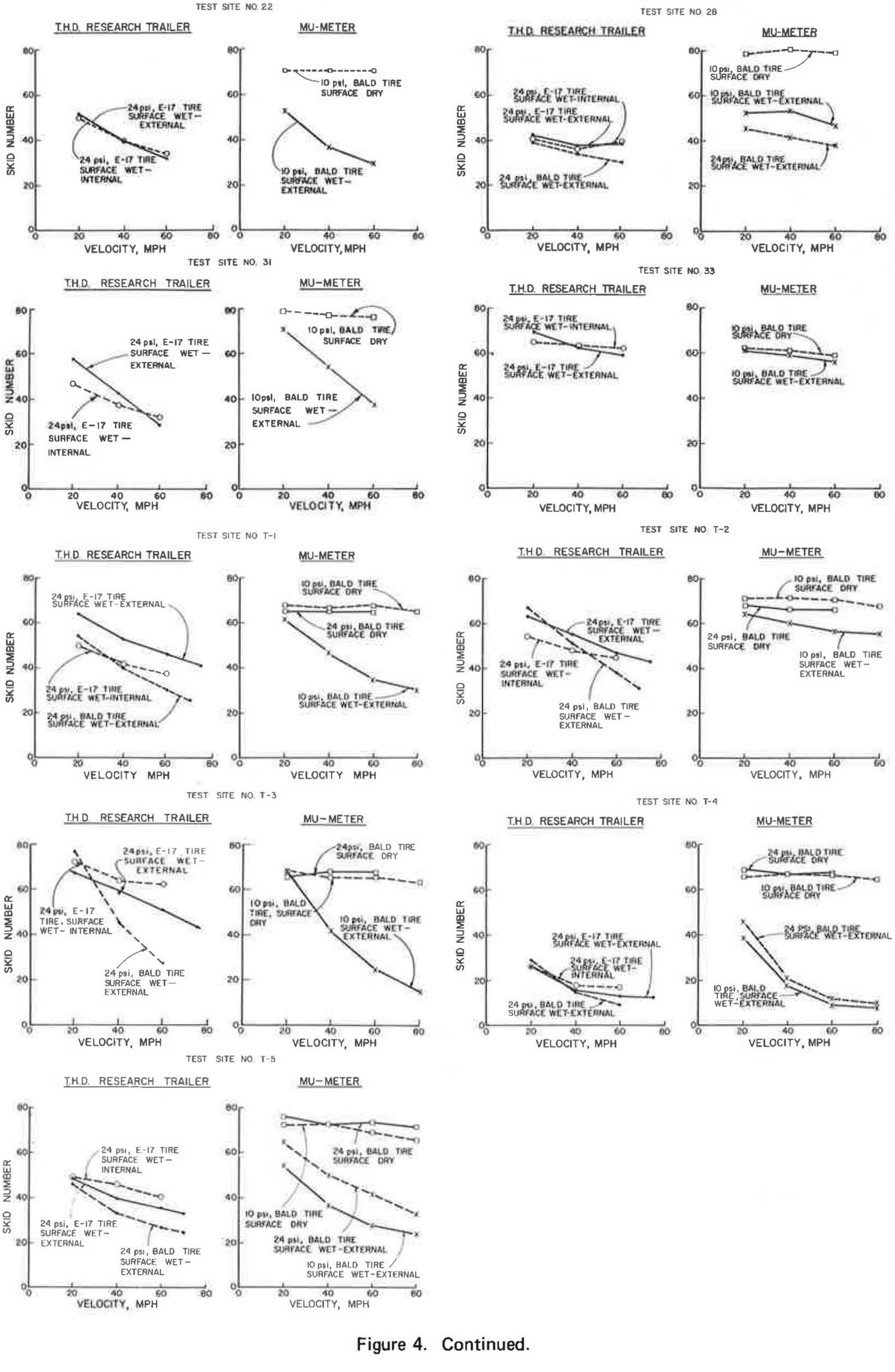

Figure 4. Continued.

both the Mu-meter (smooth tire) and the trailer (E-17 tire) at 20 mph; however, much lower values were obtained at higher speeds (Fig. 3b). This is to be expected when consideration is given to the fact that the Mu-meter operates in the cornering-slip mode, whereas the trailer operates in the skid mode. Thus, higher friction values are expected in the cornering-slip mode if other conditions are maintained constant.

The use of a treaded tire on the trailer will generally provide sufficient drainage at high speeds to increase the friction to that of an instrument operating in the cornering-slip mode with a smooth tire. At the lower speeds, however, drainage effects are reduced and the overriding effects of the cornering-slip mode prevail; thus, the Mu-meter records slightly higher friction values (Figs. 3a and 3b). It must be remembered, however, that these conclusions are specific and will not necessarily hold for all surface types, equipment, variables, and environmental conditions. For example, the curves shown in Figure 3c for test conditions 3 and 4 differ appreciably when the average curves represent only five surfaces.

The friction number-velocity data given in Table 3 are plotted with respect to individual and surfaces shown in Figure 4. From these figures, effects of the different

TABLE 5

STATISTICAL COMPARISONS OF FRICTION NUMBERS OBTAINED AT VARIOUS SPEEDS AND TEST CONDITIONS

Variables[a]		Number of Comparisons	Figure Number	Regression Line	Correlation Coefficient	Coefficient of Determination	Standard Deviation
Y	X						
$SN_{20}(3)$	$SN_{20}(4)$	15	5a	Y = -6.38 + 1.05X	0.86	0.75	6.55
$SN_{40}(3)$	$SN_{40}(4)$	15	5a	Y = 14.66 + 0.65X	0.78	0.61	8.20
$SN_{60}(3)$	$SN_{60}(4)$	15	5a	Y = 20.13 + 0.49X	0.73	0.54	8.58
$SN_{20}(4)$	$SN_{20}(6)$	10	5b	Y = 26.35 + 0.60X	0.94	0.89	3.68
$SN_{40}(4)$	$SN_{40}(6)$	10	5b	Y = 9.20 + 1.01X	0.92	0.84	7.05
$SN_{60}(4)$	$SN_{60}(6)$	10	5b	Y = -4.02 + 1.42X	0.96	0.91	6.45
$SN_{20}(3)$	$SN_{20}(6)$	10	5c	Y = 17.67 + 0.70X	0.86	0.74	7.30
$SN_{40}(3)$	$SN_{40}(6)$	10	5c	Y = 19.44 + 0.71X	0.81	0.65	8.29
$SN_{60}(3)$	$SN_{60}(6)$	10	5c	Y = 21.32 + 0.63X	0.76	0.58	7.93
$SN_{20}(3)$	$SN_{20}(2)$	15	5d	Y = 9.81 + 0.87X	0.92	0.85	4.97
$SN_{40}(3)$	$SN_{40}(2)$	15	5d	Y = 5.62 + 0.88X	0.93	0.87	4.68
$SN_{60}(3)$	$SN_{60}(2)$	15	5d	Y = 1.25 + 0.92X	0.93	0.87	4.60
$SN_{20}(1)$	$SN_{20}(4)$	15	5e	Y = 59.26 + 0.21X	0.29	0.09	7.23
$SN_{40}(1)$	$SN_{40}(4)$	15	5e	Y = 64.10 + 0.15X	0.30	0.09	7.19
$SN_{60}(1)$	$SN_{60}(4)$	15	5e	Y = 67.13 + 0.08X	0.22	0.05	6.97
$SN_{20}(4)$	$SN_{20}(7)$	7	5f	Y = 4.67 + 0.87X	0.93	0.86	4.79
$SN_{40}(4)$	$SN_{40}(7)$	7	5f	Y = 3.34 + 0.90X	0.94	0.89	7.17
$SN_{60}(4)$	$SN_{60}(7)$	7	5f	Y = -1.06 + 0.99X	0.96	0.92	7.34
$SN_{20}(1)$	$SN_{20}(5)$	9	5f	Y = 30.64 + 0.58X	0.59	0.34	4.46
$SN_{40}(1)$	$SN_{40}(5)$	9	5f	Y = 20.62 + 0.73X	0.63	0.40	4.31
$SN_{60}(1)$	$SN_{60}(5)$	9	5f	Y = 25.47 + 0.65X	0.63	0.39	3.71

[a]SN = skid or slip number, subscript indicates speed in mph, and numbers in parentheses indicate test conditions.

TABLE 6

STATISTICAL COMPARISONS OF GRADIENTS, PERCENTAGES OF GRADIENTS, AND FRICTION NUMBERS OBTAINED WITH THE VARIOUS TEST CONDITIONS

Variables[a] Y	Variables[a] X	Number of Comparisons	Figure Number	Regression Line	Correlation Coefficient	Coefficient of Determination	Standard Deviation
G(3)	G(6)	10	6a	Y = 0.24 + 0.24X	0.54	0.29	0.11
G(4)	G(3)	15	6b	Y = -0.04 + 1.37X	0.58	0.33	0.28
G(4)	G(6)	10	6c	Y = -0.02 + 0.87X	0.65	0.42	0.30
PG(3)	PG(6)	10	7a	Y = 5.61 + 0.51X	0.75	0.56	7.84
PG(4)	PG(3)	15	7b	Y = -7.68 + 1.47X	0.73	0.53	17.05
PG(4)	PG(6)	10	7c	Y = -32.34 + 1.54X	0.92	0.84	11.84
$SN_{40}(6)$	G(6)	10	8a	Y = 34.00 + 7.21X	0.13	0.02	15.83
$SN_{40}(6)$	1nG(6)	10		Y = 40.23 + 3.20 1nX	0.10	0.01	15.90
$SN_{40}(4)$	G(4)	15	8b	Y = 61.25 - 29.66X	0.65	0.42	11.92
$SN_{40}(4)$	1nG(4)	15		Y = 37.07 - 7.41 1nX	0.74	0.56	10.45
$SN_{40}(3)$	G(3)	15	8c	Y = 42.18 + 7.29X	0.08	0.01	13.12
$SN_{40}(3)$	1nG(3)	15		Y = 48.18 + 3.09 1nX	0.12	0.01	13.06
$SN_{40}(6)$	PG(6)	10	9a	Y = 63.08 - 0.57X	0.62	0.39	12.48
$SN_{40}(6)$	1nPG(6)	10		Y = 119.58 - 21.96 1nX	0.61	0.37	12.65
$SN_{40}(4)$	PG(4)	15	9b	Y = 66.31 - 0.55X	0.87	0.75	7.78
$SN_{40}(4)$	1nPG(4)	15		Y = 72.99 - 8.69 1nX	0.82	0.68	8.90
$SN_{40}(3)$	PG(3)	15	9c	Y = 63.66 - 0.62X	0.58	0.34	10.70
$SN_{40}(3)$	1nPG(3)	15		Y = 77.93 - 9.94 1nX	0.38	0.15	12.13

[a]G = gradient (slope) at the friction speed curve between 20 and 60 mph; PG = percentages of gradient of the friction speed curve between 20 and 60 mph; SN = skid or slip number; subscript indicates speed in mph; 1n = 1ay to the base e; and numbers in parentheses indicate test conditions.

tire inflation pressures, tire-tread depths, wet or dry surface conditions, and modes used in this study can be made for individual surfaces.

In order to get a better understanding and to assist in discussing the following figures, we conducted statistical analyses on the various relationships. Results are given in Tables 5 and 6 and shown in Figures 5 through 9.

Comparisons of friction numbers obtained with various test conditions are given in Figure 5. Test results are shown in the top left of Figure 5 as obtained with each instrument operating under respective standard test conditions, i.e., trailer with E17 tire, 24 psi, and Mu-meter with smooth tire, 10 psi, with the exception that an external means was used for wetting the pavement to ensure equivalent water-film thickness. Average values, with respect to velocity are very close. Considerable data scatter exists, particularly at higher velocities; however, individual surfaces tend to maintain relative positions. The correlation coefficients decrease with increasing speed, which is expected because the relative drainage abilities of the two tires differ markedly; however, drainage also contributes to the lower correlation at higher speeds. On an average, as speed increased, the skid number became lower than the cornering-slip number. This was also borne out by the regression coefficients. At 20 mph the slip-mode measure is greater than the skid mode measure; at 60 mph the reverse is true. At the higher speeds, the relative drainage abilities of the two tires affect the friction level more than the operating mode.

Figure 5, top right, also shows Mu-meter-trailer friction comparisons; however, these comparisons differ from those shown at the top left in one respect—a smooth tire was used on the trailer. This represents an attempt to equalize the relative drainage capabilities of the test vehicles and thus get a better insight into the cornering-slip mode and skid mode comparison. Cornering-slip numbers obtained at each speed were, on an average, higher than corresponding skid numbers. This is to be expected because available friction during the slip (cornering-rolling) mode is higher than available friction in skid (sliding) mode, provided other variable factors do not exist. Also, constant and substantially higher correlation coefficients were obtained for these relationships than were obtained for those shown in Figure 5 top left. This constancy indicates that the relative drainage capabilities of the vehicles were essentially identical at the given speeds. Although on an average both methods measured a decrease in friction levels with corresponding increases in speed, the range in trailer values became smaller and the range in Mu-meter values became larger with increase in speed.

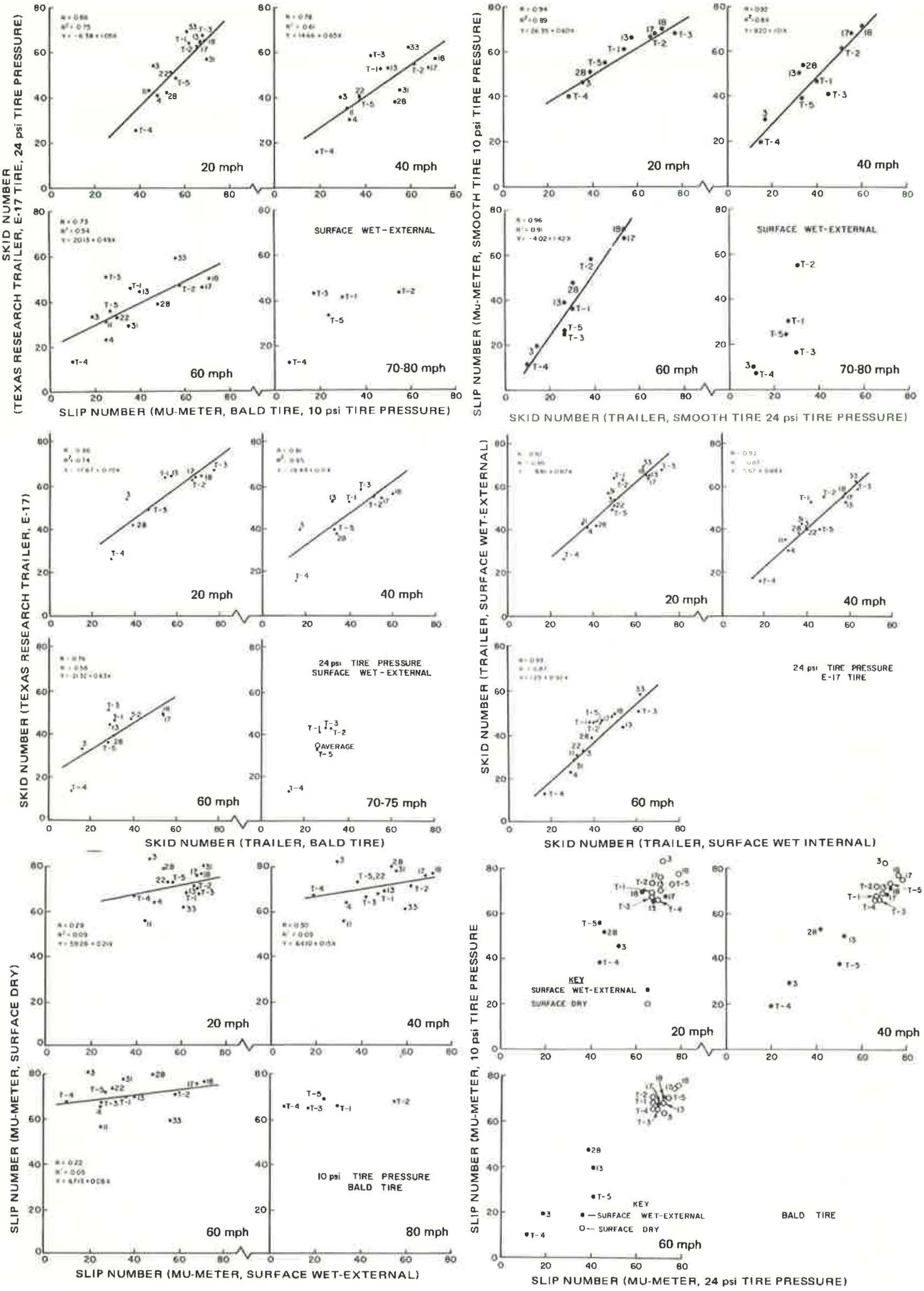

Figure 5. Friction comparisons of the various test conditions.

Various velocity comparisons of skid (trailer) tests with treaded and smooth tires are shown in Figure 5 middle left. Relative positions of the surfaces, with respect to increased velocities, are not maintained. Except for surfaces 17 and 18, the surfaces tend to deteriorate in skid resistance (with increasing velocity) at faster and more variable rates when tested with a smooth tire than when tested with the E-17 tire. This points out the relative drainage capabilities of the different tires as well as the different types of surfaces. The correlation coefficient was also lower at the higher speed.

Figure 5, middle right, shows that skid numbers obtained with the trailer at various speeds with respect to the 2 pavement wetting processes were quite similar. In general, the internal watering procedures resulted in slightly lower skid numbers at 20 mph and slightly higher skid numbers for the 60-mph tests when compared to corresponding skid tests using external watering procedures. Variations in the wetting procedures, resulting in different water-film thickness, probably account for the differences. Average 40-mph skid numbers were identical for the 15 surfaces. Consistently high correlation coefficients were obtained at each speed.

Figure 5, bottom left, shows that surface type and test velocity have little effect on dry-pavement cornering-slip number. In addition, dry-pavement slip numbers correlate poorly with wet-pavement cornering-slip numbers as evidenced by the extremely low correlation coefficients.

Limited data, comparing cornering-slip numbers obtained with tire inflation pressures of 10 and 24 psi, are shown in Figure 5, bottom right. Tire inflation effects were negligible. Correlation coefficients were high when the surfaces were tested in the wet condition. Although lower correlation coefficients were obtained when the surfaces were tested in the dry condition, all the surfaces were grouped rather closely together as far as cornering-slip number variations are concerned, thus rendering correlation somewhat meaningless in this comparison.

Comparisons of friction-velocity gradients obtained from 20- to 60-mph tests on the various surfaces are shown in Figure 6. Figure 6a shows that steeper gradients were obtained on most surfaces with the smooth tire on the trailer than with the E-17 tire. Also, the range in gradients obtained with the smooth tire was greater than that obtained with the E-17 tire. These results indicate that different types of surfaces vary appreciably in ability to drain water from under a tire. Similar conclusions can be drawn from test results shown in Figure 6b. Although smooth-tire tests were taken with the Mu-meter in this case, the range and magnitude of the gradients were likewise greater than those obtained with the treaded tire. Figure 6c shows that the test mode also influences gradient. Surfaces that had steeper gradients when tested with the trailer were suspected as having higher microtexture (although this was not measured). Microtexture would tend to heat up and melt to a limited degree the sliding rubber, thus providing additional lubrication and resulting in lower available friction. This would not be the case with the "rolling" tire on the Mu-meter. Correlation coefficients obtained in these comparisons were not very high.

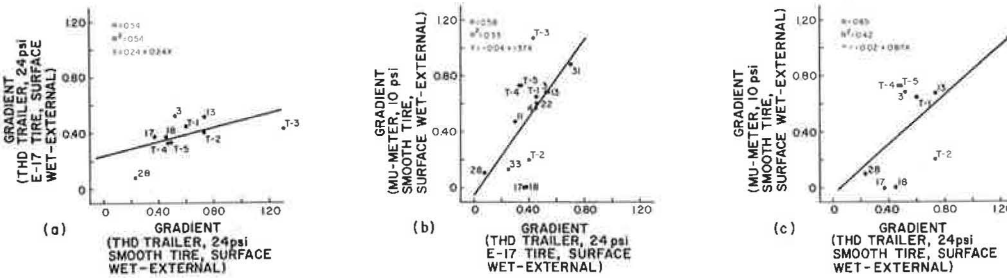

Figure 6. Comparison of friction-velocity gradients taken at 20 to 60 mph.

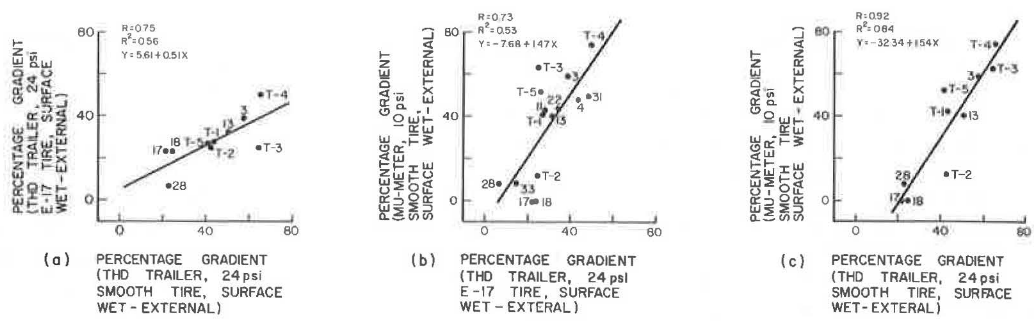

Figure 7. Comparisons of friction-velocity percentage of gradients taken at 20 to 60 mph.

Comparisons of friction number-velocity percentage of gradients obtained from 20- to 60-mph tests on the various surfaces are shown in Figure 7. Similar conclusions can be drawn from data shown in Figures 7a and 7b as were drawn from those shown in Figures 6a and 6b. However, Figure 7c shows a much higher correlation between percentage of gradient as obtained with smooth tires on the Mu-meter and trailer. Figure 6c does not show nearly as high a correlation.

Comparisons of 40-mph friction numbers and 20- to 60-mph friction number-velocity gradients for the various surfaces are shown in Figure 8. The trailer plots shown in Figures 8a and 8c do not indicate that skid number and gradient are negatively related, although the band of values is quite wide. The Mu-meter tests indicate that to some extent higher friction surfaces are associated with flatter gradient surfaces. Such was not evidenced from the trailer tests.

Comparisons of 40-mph friction numbers and 20- to 60-mph friction number-velocity percentage gradients for the various surfaces are shown in Figure 9. A negative relationship is indicated for each test condition, with the best relationship obtained using the Mu-meter. This indicates that surfaces with high 40-mph friction numbers tend to degrade less in available friction with increased speed than do surfaces with low 40-mph friction numbers. Points positioned to the right of the best-fit line represent surfaces that are deceptive, i.e., for a given friction number at 40 mph, the amount of the available friction at 60 mph is quite low when compared with surfaces positioned to the left of the line at the given 40-mph friction number.

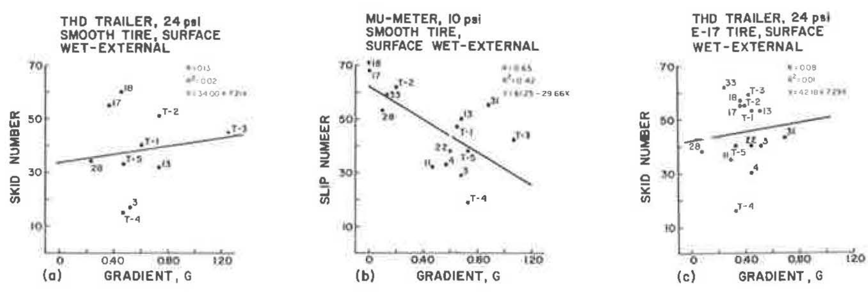

Figure 8. Comparison of friction numbers at 40 mph and friction-velocity gradients at 20 to 60 mph.

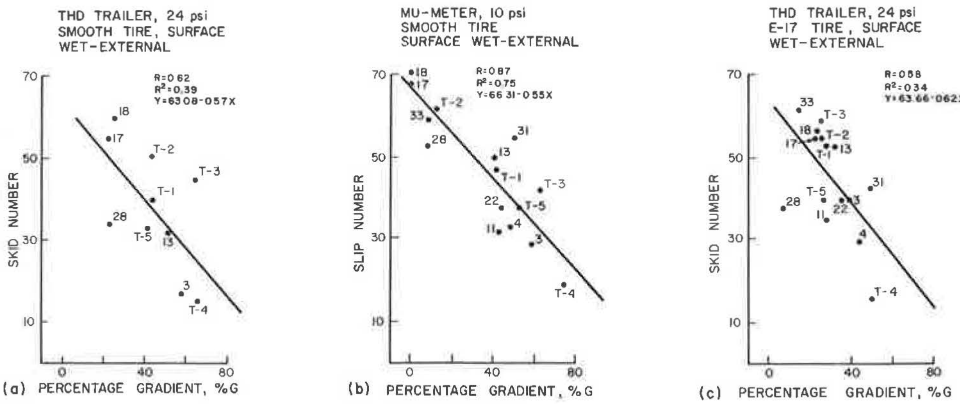

Figure 9. Comparison of friction numbers at 40 mph and friction-velocity percentage of gradients at 20 to 60 mph.

CONCLUSIONS

Based on the test procedures, equipment, and environmental conditions associated with the collection of data presented in this report the following conclusions appear to be warranted.

1. Good correlations were found to exist between the Mu-meter and the Texas Highway Department skid trailer at speeds of 20, 40, and 60 mph provided that both instruments utilized treadless or smooth tires and further provided that the surfaces being tested were wet to similar degrees. Correlation coefficients ranged from 0.92 to 0.96.
2. Comparisons made in the wet condition with the trailer using ASTM E-17 treaded tires and the Mu-meter using smooth tires yielded correlation coefficients that ranged from 0.86 to 0.75 for speeds from 20 to 60 mph.
3. Analysis of the data indicates that the relative drainage capabilities of the smooth and treaded tires becomes highly critical for certain surfaces (pavements) with limited rugosity.
4. The external and internal watering systems used were not equally efficient. At higher speeds (60 to 80 mph) the internal watering system used by the trailer becomes measurably less effective, probably due to splash and wind effects.
5. For the water-film thickness used in this study (approximately 0.020 in.), friction measurements on surfaces with macrotexture greater than about 0.025 in. were essentially the same for smooth and treaded tires. Although numerical values of microtexture were not available for these comparisons, the surfaces were considered to have about equal microtexture.
6. Numerous tests on clean, dry surfaces with the Mu-meter indicated little variation with speed or surface type with all surfaces exhibiting high values. A similar statement can be made for locked-wheel stops on clean, dry surfaces.
7. Comparisons of the percentage of gradients of the friction-velocity curves when Mu-meter and the trailer were operating with smooth tires gave a correlation coefficient of 0.92, whereas in a similar comparison made between the Mu-meter (smooth tires) and the trailer with E-17 treaded tires the correlation coefficient was much lower.
8. Variation of the Mu-meter tire pressure produced little effect on tests made on wet surfaces.

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conclusions expressed in this paper are those of the authors and not necessarily those of the sponsors.

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TWO LABORATORY METHODS FOR EVALUATING SKID-RESISTANCE PROPERTIES OF AGGREGATES

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In this paper a description and evaluation of two methods are given for determination of aggregate skid-resistance properties in the laboratory. In the first method, pavement samples manufactured from the aggregate to be evaluated are placed in a circular track and subjected to wear from small-diameter pneumatic tires. No abrasive or water is used, and pavement specimens can usually be brought to terminal polish in about 16 hours. Skid-resistance values are determined by using the British portable tester. In the second method, coarse aggregates are polished dry in a jar mill with a charge of flint pebbles. Pavement samples are made from the polished aggregates and tested for skid resistance by using the British portable tester. Polish rate is determined by exposing a number of samples of the same aggregate for different amounts of time to establish a wear-time curve. Results from the two methods are different in value level but show a linear correlation. Also, aggregates were rated in the same order in both methods. Twenty aggregates were used.

•THE research results reported here are based on a thesis (1) and two reports (2, 3) by the authors on wear and polishing properties of aggregates oriented to the pre-evaluation of aggregates in the laboratory for pavement surface course use. Objectives of the research include development of test procedures including test machines, determination of reasons for differences in performances of different aggregates, and evaluation of specific paving mixtures in the laboratory and the field.

The part of the overall research reported in this paper is the description and evaluation of two methods for determination of aggregate skid resistance in the laboratory. In the first method, pavement samples manufactured from the aggregate to be evaluated are placed in a circular track and subjected to wear from small-diameter pneumatic tires. No abrasive or water is used, and pavement specimens can usually be brought to terminal polish in about 16 hours. Skid-resistance values are determined by using the British portable tester. In the second method, coarse aggregates are polished dry in a jar mill with a charge of flint pebbles. Pavement samples are made from the polished aggregates and tested for skid resistance by using the British portable tester. Polish rate is determined by exposing a number of samples of the same aggregate for different amounts of time to establish a wear-time curve.

CIRCULAR TRACK WEAR METHOD

The Machine

A circular track machine utilizing smooth pneumatic tires (4.10/3.50-5) was constructed (Fig. 1). The machine consists of four wheels that travel over a segmented circular track (Fig. 2) and an electric motor to drive the central shaft. The circular

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Figure 1. Circular track machine.

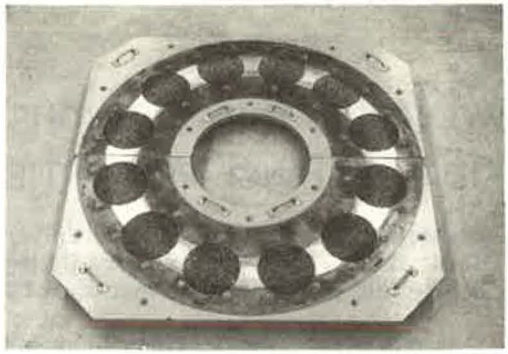

Figure 2. Removable track from circular track machine.

track contains spaces for twelve specimens. Each specimen may be adjusted individually for elevation and level. Track segments must be removed from the machine for friction measurements (Fig. 3).

The tires rotate around the 36-in. diameter track at the rate of 30 rpm generating 7,200 tire-passes per hour. We attempted to obtain accelerated machine polishing action by adjusting the tire setup so that two of the diametrically opposed wheels would toe-in while the other two would toe-out producing a scrubbing action in addition to the rolling action.

The Test Specimen

A circular specimen 6 in. in diameter was used in the wear and polishing machine circular track. Thickness may vary but must be in the range of 1 to 2 in. to be accommodated in the machine specimen holder. This type of specimen can be prepared in the laboratory or may be cored from pavements in service. Bituminous or portland cement concrete specimens can be used with equal facility. Results from bituminous specimens only are reported here.

It has been determined by several investigators (4 through 8) that the skid resistance of a bituminous pavement surface is determined primarily by the type and size of aggregate used in the surface mix. Coarse aggregate particles (greater than ⅛ in.) were found to have the primary influence even when they constituted only a moderate percentage (20 to 30 percent) of the aggregate used in the mix. The mix adopted for this research is an open-graded bituminous mix consisting primarily of aggregate greater than ⅛ in. The advantage of this type of mix is that the surface contains only one aggregate, the one to be evaluated. In addition, this type of mix has been placed on experimental surfaces by the North Carolina State Highway Commission (NCSHC); therefore, an opportunity exists for laboratory-field performance correlation. The composition of this mix is within the range of NCSHC No. 13 stone. The bitumen used for all mixes was obtained by the NCSHC from a single source of 85 to 100 penetration grade asphalt cement. Generally, the asphalt cement content of a mix was kept at the relatively low level of 6

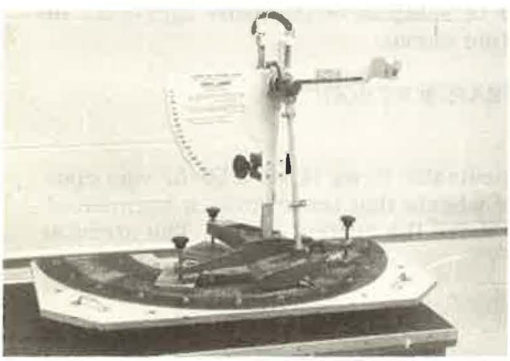

Figure 3. British portable skid-resistance tester.

percent by weight of aggregate (5.66 percent by weight of total mix). The No. 13 stone gradation has been used for all laboratory specimens prepared for this research, unless otherwise stated, and is as follows:

Sieve	Percent Passing
$\frac{1}{2}$ in.	100
$\frac{3}{8}$ in.	95
No. 4	43
No. 10	10
No. 40	4
No. 80	3
No. 200	2

Mounting and Testing

The surface of each specimen was cleaned with trichloroethylene solvent before mounting to remove, as much as possible, the asphalt surface coating and to expose the aggregate surface for acceleration of the test by the time required to wear away this asphalt.

The specimens were tested for initial skid resistance by using the British portable tester, and the track plates were fitted in place in the machine, checked for level, and bolted down. The tires were lowered to the track surface and the machine set in motion. Tire inflation pressure was kept at 20 psi and the tire-pavement contact pressure at 13 psi, resulting in a contact area of approximately 5 sq in. with an average width of 3.2 ± 0.1 in.

No abrasive powder or water was used to assist the wear and polish action. This procedure was adopted as a result of a field investigation of material on the surface of pavements across the state of North Carolina. It was found that very little loose material was on the pavement surface, and most of that found could be identified as rubber asphalt and aggregate dust from the pavement itself. It is well known that pavements are dry most of the time.

Introduction of $\frac{1}{2}$-in. toe of wheel mounting from axle centers accelerated the test so that first reading could be taken after 1 hour instead of 20 hours without toe, and essentially terminal polish could be reached in 16 hours instead of 2 weeks as before. Friction measurements were made at 0, 1, and 2 hours and then every 2 hours up to 16 hours.

Initially, it was hoped that the accelerated test sequence could be limited to 8 hours, and this was the case for the first three series of 12 specimens. Beginning with the fourth series, some specimens did not show essentially terminal polish until after 16 hours' exposure, and the 16-hour test duration was adopted for all remaining series.

At the end of each time interval, the machine was stopped and the specimens were tested for frictional skid resistance by using the British portable tester as shown in Figure 3.

Circular Track Test Results

Three replicate specimens were made from each of the 20 aggregates given in Table 1. Twelve specimens representing four aggregate types were placed in the track for each machine run series.

To provide a standard for comparing one test series with another, it was decided to include three replicate specimens made of a stock aggregate as control specimens with every series to be tested. The stock aggregate selected to be used as control was a quartz-disc muscovite gneiss locally available in any quantity desired, and one that, in preliminary testing, showed reasonably gradual wear and polish. A supply of this aggregate given in Table 1 as GN-1 was stockpiled. Inclusion of the control aggregate specimens reduced machine efficiency to 75 percent of specimen capacity.

Results of skid-resistance testing on all seven series are reported in terms of average British portable numbers (BPN) versus hours of exposure as given in Table 2. De-

TABLE 1

PHYSICAL PROPERTIES OF THE SAMPLE AGGREGATES

Number	General Classification	Grading C Los Angeles Wear Loss (percent)[a]	Bulk Specific Gravity[a]	Water Absorption (percent)[a]
LS-1	Limestone	18	2.85	0.30
LS-2	Limestone	25	2.87	0.40
LS-3	Limestone	46	2.47	3.35
LS-4	Limestone	29	2.95	0.30
GT-1	Granite	36	2.79	0.31
GT-2	Granite	63	2.67	0.42
GT-3	Granite	51	2.65	0.50
GT-4	Granite	41	2.66	0.50
GN-1	Granite gneiss	29	2.67	0.41
GN-2	Granite gneiss	52	2.68	0.6
GN-3	Granite gneiss	48	2.71	0.55
GL-1	Gravel	42	2.64	0.30
GL-2	Gravel	43	2.78	1.01
SL-1	Slate	17	2.78	0.32
SL-2	Slate	24	2.78	0.33
RH-1	Rhyolite	27	2.67	0.30
TR-1	Traprock (diabase)	15	2.77	0.30
SS-1	Sandstone (arkose)	N.A.	2.66	2.55
SP-1	Expanded glass[b]	23.3	2.05	2.4
SO-1	Expanded slate[b]	40	1.58	3.5

[a] Determined by the Materials Laboratory of the North Carolina State Highway Commission.
[b] Provided by manufacturer.

TABLE 2

BPN DATA CIRCULAR TRACK SPECIMENS

Series	Aggregate	Circular Track Machine Hours[a]																
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	GN-1	56	51	51	48	48	47	48	46	45								
	SP-1	62	54	52	47	49	51	47	48	48								
	SO-1	60	57	52	50	52	53	51	53	53								
	SS-1	62	59	63	64	63	59	60	–	61								
2	GN-1[b]	56	51	51	48	48	47	48	46	45								
	SL-1	51	50	49	48	49	48	46	48	46								
	LS-3	52	47	45	46	44	47	44	45	44								
	LS-2	55	49	46	46	44	45	42	43	42								
3	GN-1	62	51	51	49	49	47	47	45	45								
	GL-1	57	51	48	49	47	46	46	44	43								
	GT-3	59	51	53	51	51	47	48	47	45								
	LS-1	55	44	42	40	41	40	38	38	37								
4	GN-1	58	49	52	51	52	44	45	46	43	41	43	41	42	39	39	39	39
	TR-1	59	48	51	49	49	45	43	45	44	40	41	40	39	38	39	39	37
	SL-2	59	51	56	52	52	48	45	45	45	46	43	44	43	44	43	43	43
	RH-1	59	48	53	51	47	45	45	43	42	42	43	43	40	40	39	38	39
5	GN-1	57	46	48		45		43		44		42		41		44		43
	GN-3	57	47	48		45		44		44		40		41		42		42
	GL-2	55	48	48		46		44		44		44		42		44		44
	GT-2	58	50	49		48		44		44		43		43		44		43
5	GN-1	53		46		46		46		48		44		44		47		47
	GT-1	52		47		47		44		45		43		45		46		45
	GT-4	57		49		48		47		45		45		45		48		46
6	LS-4	51		45		46		44		42		43		42		44		44
7	GN-1	55	50	47		46		46		45		45		44		44		44
	GN-2	57	49	48		46		46		45		45		44		44		42
	RH-1[c]	50	47	45		44		43		43		43		42		42		41
	GT-1[c]	49	48	45		45		46		44		45		42		42		42
	GN-1[d]	56.5	50	49.5	49	47.5	46	46	45.5	45		43.5		43		43		43

Note: Each BPN value is an average of 15 BPT pendulum swings made on three specimens of the same aggregate.

[a] Each hour of machine wear exposure represents 1,800 revolutions and 7,200 tire passes.

[b] Data same as for GN-1 in series 1 because one specimen of three in each of the two series was badly shoved. Other two specimens had similar data; therefore, they were averaged together.

[c] These specimens were cored from NCSHC test pavements with thin overlay open-graded bituminous mixes.

[d] Average of measurements on 19 specimens made from aggregate GN-1 and used for control.

viations from the average BPN were within ±3 numbers for individual specimens of a given aggregate in any measurement with amount of spread decreasing with increased time of exposure. All BPN measurements were made by the same operator.

Discussion of Data Curves

Testing results given in Table 2 have been plotted selectively and are shown in Figures 4 and 5 to illustrate change in BPN versus exposure time in the wear machine for the various aggregates tested. Three curve patterns emerged, and these are shown in Figure 6.

The hyperbolic curve relationship is not unfamiliar in reports by skid-resistance investigators who have undertaken to study the variation in pavement skid-resistance characteristics with increasing vehicle passes (7, 8, 9). What is lacking at this time is a quantitative correlation between number of vehicle passes on actual pavements and the laboratory machine tire passes over the pavement specimens. This correlation will require simultaneous field and laboratory testing on identical paving surfaces. However, the pavement specimens that have been tested in the laboratory exposure (Fig. 5) were brought to an ultimate state of wear and polish approximating that of laboratory specimens, and it has been assumed that the skid numbers measured toward the termination of the machine exposure may represent conditions on pavements subjected to prolonged traffic action.

An examination of the curves shown in Figures 4 and 5 and of the other data given in Table 2 leads to the following observations:

1. All specimens, regardless of aggregate used, showed satisfactory BPN values when surfaces were cleaned before actual exposure began;
2. Most specimens exhibited a rapid loss of BPN, or high rate of polish, during the first 4 to 6 hours of exposure in the CT wear machine, but the loss, representing rate of polish, varied depending on the aggregate;
3. The specimens did polish without the addition of water or abrasive to help the polishing action;
4. The wear machine does separate aggregates by performance for the exposure used;

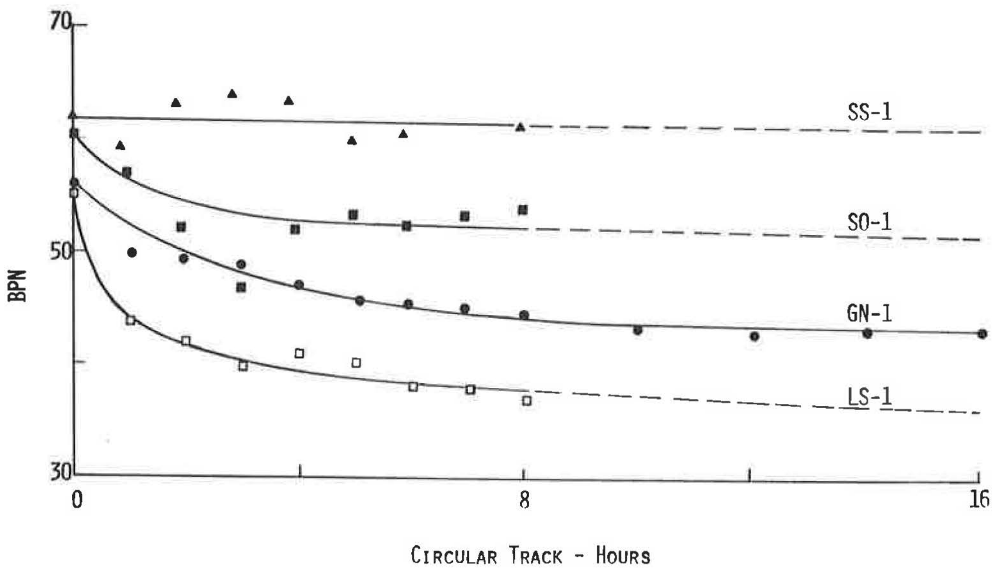

Figure 4. BPN versus circular track polishing time curves for GN-1, SP-1, SO-1, and SS-1.

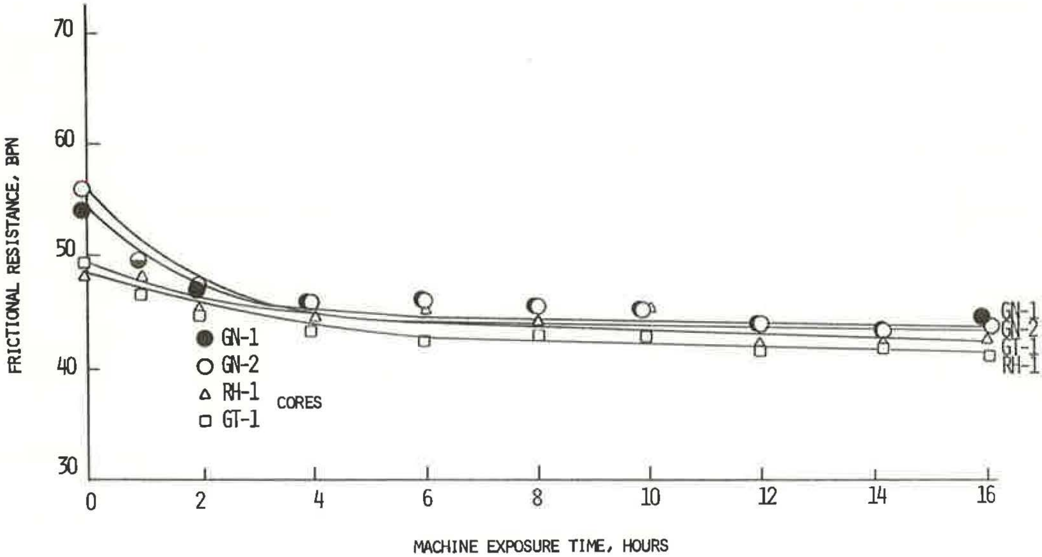

Figure 5. BPN versus circular track polishing time curves for GN-1, GN-2, RH-1, and GT-1.

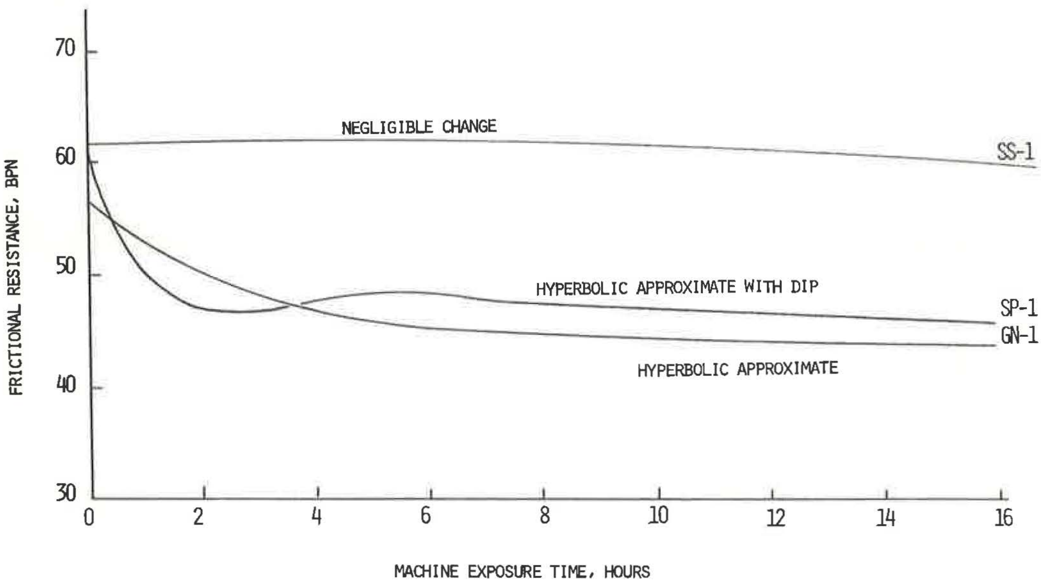

Figure 6. Typical BPN versus wear exposure time curves.

5. Most curves had flattened out by 8 to 12 hours of exposure, indicating very little additional polish or loss of skid resistance with additional exposure;
6. Field and laboratory specimens made with the same aggregate showed similar wear and polishing characteristics in the wear machine exposure; and
7. A dip or loss of BPN followed by recovery is indicated in points for many of the plotted curves.

The reason for this dip in curve is not actually known. It may be speculated that some initial surface polish occurs followed by an actual loss of surface particles or that some asphalt film is being spread and then worn away. The presence of the dip does not appear to influence the final leveling out, or equilibrium position.

Serafin (10) has reported the occurrence of this dip followed by recovery phenomenon on several field bituminous pavements in Michigan, but no detailed explanation is offered.

Rating of Sample Aggregates

To obtain a meaningful comparison between the 20 sample aggregates, we established a standard curve. BPN values obtained for each of the GN-1 specimens used in the seven series were averaged. The result is shown in Figure 7 as the average curve for the GN-1 aggregate. This curve has been used as a standard for comparing the values of all the test series curves by adjusting the GN-1 curve in each series to this average curve and, as a consequence, adjusting all other curves in the same series accordingly.

As critical skid-resistance characteristics of an aggregate occur when high-to-ultimate polish is attained, values of each curve for the aggregates in series 1 through 7 (Table 2) were compared to the "standard" average curve shown in Figure 7 at the points indicating the end of the 8th and the 16th hour of machine exposure. (For series 1 through 3, values beyond 8 hours were extrapolated.) At each of these two points, the BPN difference between the standard curve and any other curve was added or subtracted in order to bring BPN to the standard curve values. This difference has been designated as an adjusting factor to be added or subtracted according to sign from the standard curve value at the point in time appropriate. In the seven series that have been tested

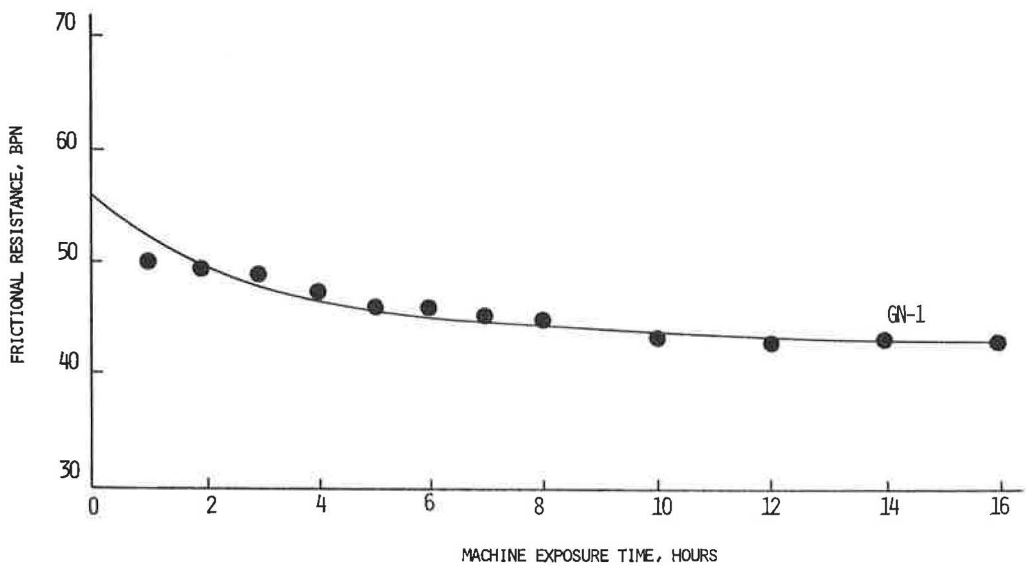

Figure 7. BPN average for seven test series on circular track for GN-1.

and compared, adjusting factor values varied from -2.0 to +2.5 BPN at the end of the 8th hour and from -3.0 to +3.0 BPN at the end of the 16th hour.

This procedure made it possible to compare all 20 aggregates used in this study to one another and to give each aggregate a rating as to its skid-resistance characteristics relative to the other aggregates that have been tested by the same procedure.

Adjusted and unadjusted BPN values for 8 and 16 hours, adjusting factors, and a relative rating for the 20 aggregates used in this study are given in Table 3. Ratings are shown in Figure 8. Generally, the same rating would have been obtained whether the adjusted BPN values at the end of 8 hours or at the end of 16 hours were considered. Because the 16th-hour BPN value for any of the curves being considered is closer to the ultimate state of polish (where the curves generally have leveled off), this value has been used as the criterion for the relative aggregate rating. When two or more aggregates had the same adjusted BPN value at the end of the 16th hour of wear and polish, the aggregate with the higher BPN value at the end of 8 hours of wear and polish has been given the higher rating.

The aggregates tested in this study fall approximately into three major categories: (a) those with high BPN, (b) those with medium-range BPN, and (c) those with low BPN values (Table 3 and Fig. 8). Most of the aggregates fall in the medium range. Some investigators have found that the BPN values given each aggregate result in a valid rating as to the aggregate skid-resistance performance although the BPN value separation may be narrow (11).

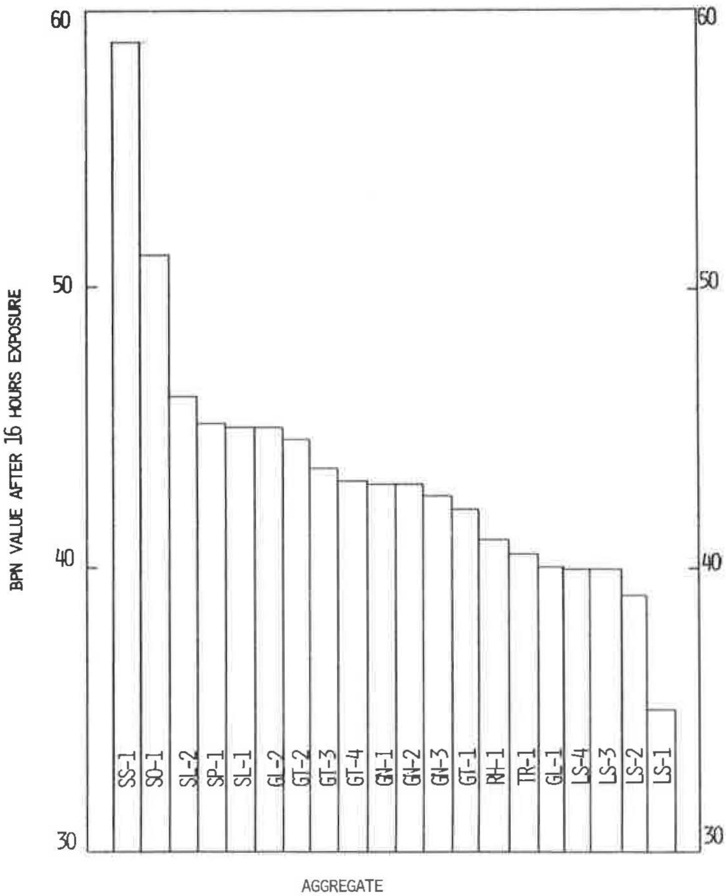

Figure 8. Skid-resistance rating of selected aggregates.

TABLE 3

RELATIVE RATING OF SKID-RESISTANCE CHARACTERISTICS OF SAMPLE AGGREGATES

Aggregate	BPN Values (8 hours)			BPN Values (16 hours)			Relative Rating
	Unadjusted Curve Value	Adjusting Factor	Adjusted Value	Unadjusted Curve Value	Adjusting Factor	Adjusted Value	
SS-1	61.5	-0.5	61.0	60.0	-1.5	58.5	1
SO-1	52.5	-0.5	52.0	52.5	-1.5	51.0	2
SL-2	44.0	2.5	46.5	43.0	3.0	46.0	3
SP-1	47.0	-0.5	46.5	46.5	-1.5	45.0	4
SL-1	47.0	-1.0	46.0	46.5	-1.5	45.0	5
GL-2	43.8	1.5	45.3	43.5	1.5	45.0	6
GT-2	43.5	1.5	45.0	63.0	1.5	44.5	7
GT-3	46.0	-1.0	45.0	44.5	-1.0	43.5	8
GT-4	46.5	-2.0	44.5	46.0	-3.0	43.0	9
GN-1	44.2	0.0	44.2	43.0	0.0	43.0	10
GN-2	44.0	-0.25	43.7	43.5	-0.5	43.0	11
GN-3	42.0	1.5	43.5	41.0	1.5	42.5	12
GT-1	45.0	-2.0	43.0	45.0	-3.0	42.0	13
RH-1	41.0	2.5	43.5	38.5	3.0	41.5	14
TR-1	41.0	2.5	43.5	37.5	3.0	40.5	15
GL-1	43.0	-1.0	42.0	41.0	-1.0	40.0	16
LS-4	43.5	-2.0	41.5	43.0	-3.0	40.0	17
LS-3	42.5	-1.0	41.5	41.5	-1.5	40.0	18
LS-2	41.5	-1.0	40.5	40.5	-1.5	39.0	19
LS-1	37.0	-1.0	36.0	36.0	-1.0	35.0	20

JAR MILL WEAR METHOD

It is generally conceded that a slippery-when-wet pavement condition occurs when the pavement surface aggregate has been polished through gradual but continuous frictional wear. It was reasoned that, if samples of loose aggregates were subjected to any reasonable gradual method of wear and polishing, the polished particles should, when tested, reflect the skid-resistance characteristics of the particular aggregate they represent. A laboratory jar mill utilizing porcelain jars was used to achieve the gradual wearing and polishing of aggregate samples. This method and the results that were obtained by using it are described in the following paragraphs.

Apparatus and Procedure

In the jar mill method, glazed porcelain jars, each having an inside diameter of approximately 9.0 in., a clear depth of 8.5 in., and a mouth opening diameter of 5.0 in., were charged with 1,000 grams of loose aggregate and 1,000 grams of ¾-in. flint pebbles for an abrasive charge. Aggregate to be used in the jars was obtained from NCSHC No. 13 stone by sieving the fraction passing the ⅜ in. and retained on the No. 4 sieve. The sieved fraction was washed and oven-dried before 1,000 ± 0.1 grams were weighed for the jar. Flint pebbles were similarly washed and dried before being charged into the jar (Fig. 9). Sealed jars were placed on the rollers of a jar mill (Fig. 10) where they were rotated at 52 rpm for the period of time required.

In experiments with this method of aggregate wear, samples were tumbled for short periods of time, from 1 to 20 hours. Results indicated that day-long increments would produce measurable changes. Periods of exposure of 0, 20, 48, 72, 96, and 120 hours were adopted for the jar mill procedure. Other abrasive charges were tried, including hard rubber balls, but flint pebbles have been found most effective.

Two rounds of tests were completed for each of the aggregates used. A round for one aggregate consisted of a control sample and separate 1,000-gram samples for each of the five exposure increments. For example, the jar containing the 20-hour sample was removed after 20 hours and processed, the 48-hour sample at 48 hours, and so on.

At the end of all wearing cycles, the aggregate sample was sieved mechanically over ⅜-in. and Nos. 4, 8, 16, 30, 50, 100, and 200 sieves for gradation. Aggregate retained on the No. 4 sieve was washed, oven-dried, and weighed to determine loss from the

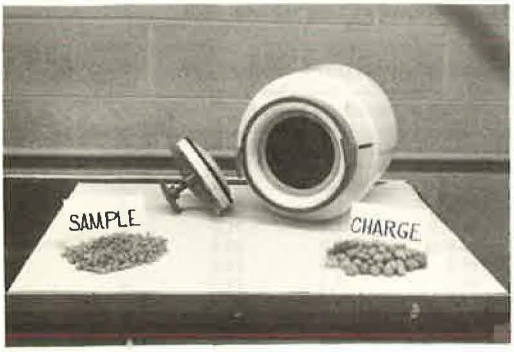

Figure 9. Jar, aggregate sample, and charge.

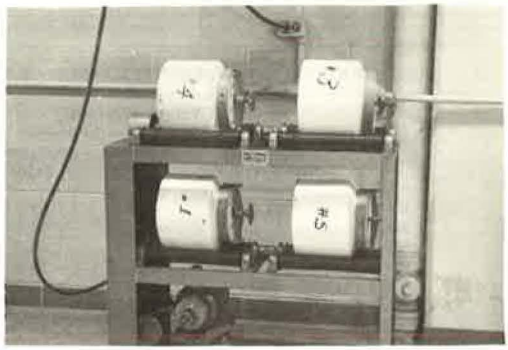

Figure 10. Jar mill machine.

original 1,000-gram weight. The No. 4 material was saved for incorporation into a laboratory pavement specimen.

Wear Test Results

Eight of the aggregates that had been tested by the circular track polishing machine method (Table 1) were selected for preliminary testing by the jar mill method. They were selected to represent differing types of aggregates that were used in this study. Results of the jar mill testing for the eight aggregates selected, computed as percentage of wear loss versus number of hours of grinding, are shown in Figure 11. Each percentage value is the average of two test results for each aggregate and wearing period.

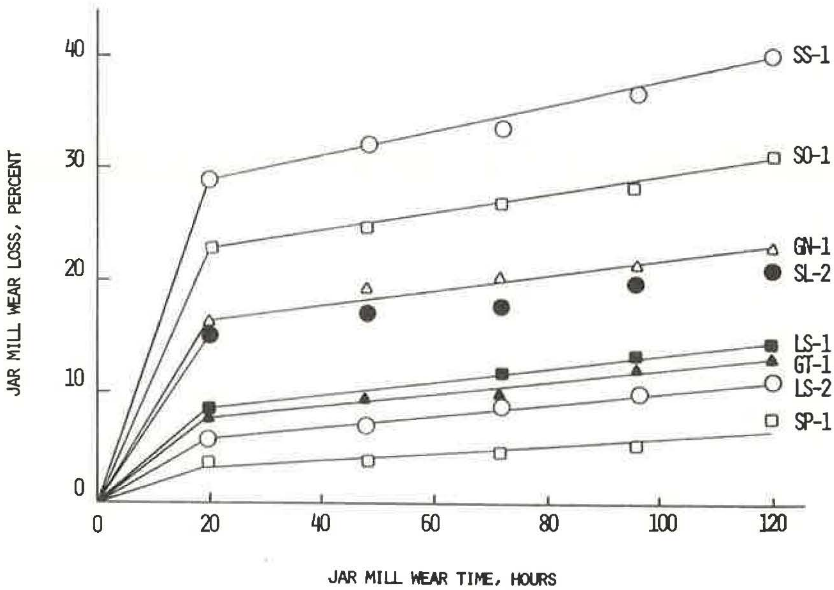

Figure 11. Jar mill wear loss versus wear time.

Skid-Resistance Test Specimen

The worn material retained on the No. 4 sieve from each of the aggregate samples was used for molding a 6-in. diameter pavement specimen. Each specimen was made with two unlike surfaces: One surface was made of the material worn in the jar mill, whereas the opposite surface of the same specimen was made from the same type and size of unworn aggregate. Each specimen had the mix gradation given in Table 4. Specimens were bound by using 5 percent by weight of aggregate of an 85 to 100 penetration asphalt cement. Eighty specimens were manufactured representing two complete series of tests for each of eight aggregates. All specimens were made by the same operator.

TABLE 4

JAR MILL PAVEMENT SPECIMEN GRADATION

Sieve		Percent Retained on Each Sieve	Accumulative Percent Retained
Passing	Retained		
$^3/_8$ in.	No. 4	88.0	88.0
No. 30	No. 50	8.5	96.5
No. 50	No. 100	1.0	97.5
No. 100	No. 200	0.5	98.0
No. 200	Pan	2.0	100.0

The final step in specimen preparation was the removal of asphalt cement from exposed aggregate surfaces by carefully cleaning with trichloroethylene. Aggregate fines were also removed from the surfaces during this cleaning procedure.

Measurement Results and Discussion

Each of the 80 specimens was tested on both surfaces for skid resistance by one operator using the British portable tester in accordance with ASTM Designation E 303-66T. Two sets of readings were taken on each specimen face at 90-deg orientation. Average results are shown in Figure 12.

Comparison of data shown in Figure 12 and in Figure 8 reveals that, for the eight aggregates being compared, both the jar mill method and the circular track method produced the same rating of aggregate for skid-resistance characteristics and the same relative grouping into high, medium, and low categories. As would be expected, initial BPN values of unworn aggregate are the same for each aggregate in each test method, but BPN values of worn aggregate are higher for the jar mill method than for the circu-

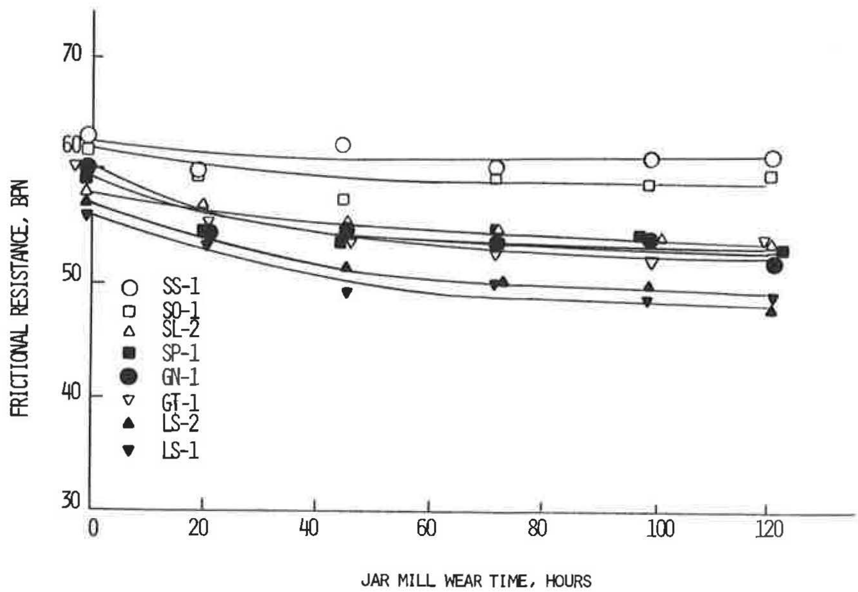

Figure 12. BPN versus jar mill wear time.

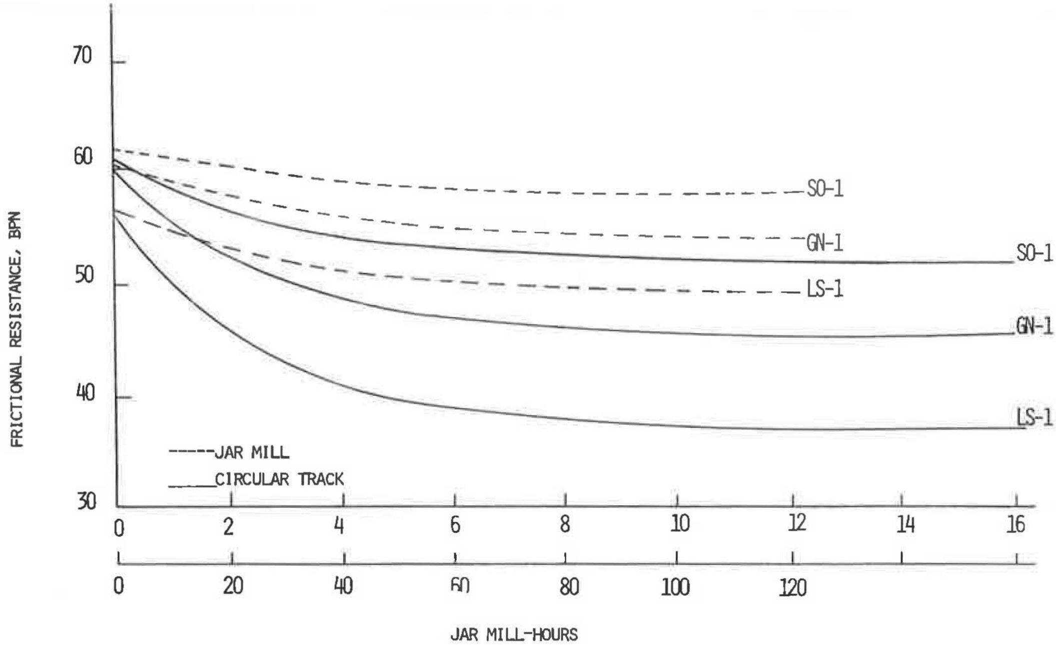

Figure 13. Comparison of typical skid resistance curves for circular track and jar mill wear exposures.

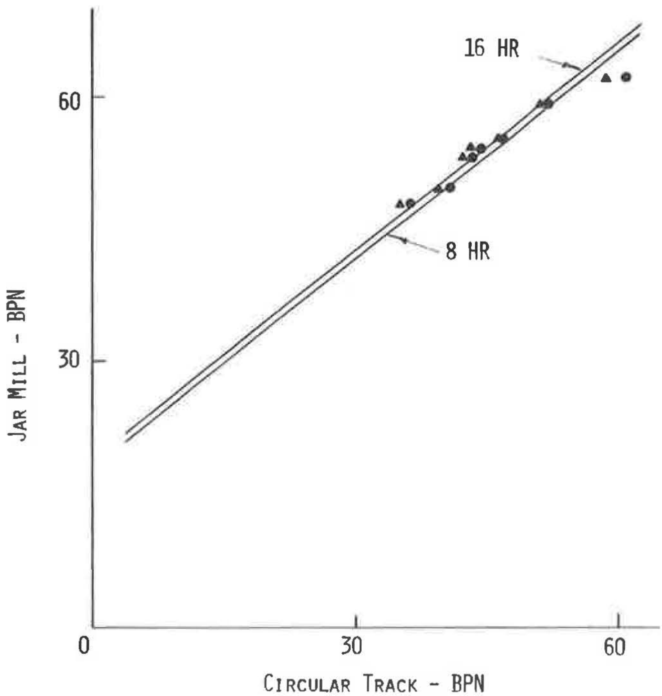

Figure 14. Correlation between BPN values for circular track and jar mill methods.

lar track method. The two methods are comparable but not equal as shown in Figures 13 and 14. Figure 14 shows that fairly good linear correlations are developed between the two methods for two-time increments of the circular track method. Particularly important is the lack of a correlation between percentage wear loss shown in Figure 11 and terminal skid resistance after wear shown in Figure 12. This lack of correlation seems to eliminate the possibility of using a wear loss test as a means of pre-evaluating aggregates for skid resistance.

ACKNOWLEDGMENTS

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The opinions, findings, and conclusions reported in this paper are those of the authors and not necessarily those of the sponsors or the Federal Highway Administration.

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FACTORS INFLUENCING AGGREGATE SKID-RESISTANCE PROPERTIES

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Aggregates that had been rated in laboratory paving mixtures for wear and polishing resistance have been examined to determine factors that influenced differences in laboratory skid-resistance properties observed. Aggregates have been examined for physical properties such as specific gravity, absorption, and surface texture; for Los Angeles wear; for acid-insoluble residue percentage and gradation in the case of carbonate aggregates; and petrographically for mineral composition, grain shape, grain size and distribution, and hardness of minerals. An attempt has been made to relate the properties determined to observed laboratory skid-resistance performance. No general correlations were observed between physical properties of aggregates and laboratory skid resistance. Correlations were observed, however, within some petrographic groups. For granite aggregates, higher Los Angeles wear loss indicated higher skid resistance. For synthetic aggregates, high absorption and surface capacity seemed to correlate with higher skid resistance. The acid-insoluble residue percentages for the four carbonate aggregates examined indicated that skid resistance improved with increased residue and that sand-size residue probably is more important than total residue. A general correlation was found between the petrographic properties and the skid resistance of any given aggregate: skid resistance was higher for aggregates having mixed composition of hard and soft minerals than for aggregates consisting predominantly of minerals of the same type having the same hardness.

•THIS paper reports on factors that influence the laboratory skid resistance of paving aggregates. The study is part of an overall research program on laboratory and field determination of wear and polishing properties of aggregates as these properties affect the skid resistance of pavements.

Prior to this study, two laboratory methods had been developed for predetermining skid-resistance properties of aggregates subjected to wear and polishing, the circular track and the jar mill methods (2, 3). Different aggregates were polished to different terminal skid-resistance levels, and there was a linear correlation between methods even though terminal polish levels were not the same for a given aggregated in both methods. All aggregates tested, however, had the same relative rating in both test methods. Skid resistance was measured by using the British portable tester (BPT) and was recorded in British portable numbers (BPN) in accord with ASTM Designation E 303-69.

The purpose of this study was to discover why different aggregates behave differently under the same wear and polishing exposures. Twenty aggregates that had been evaluated by using the circular track and jar mill methods were subjected to test for physical properties, for insoluble residue in the case of carbonate rocks, and for petrographic properties. These various properties have been compared to wear and polish results for possible correlation. Figure 1 shows the steps in the investigation procedure.

Sponsored by Committee on Surface Properties-Vehicle Interaction and presented at the 50th Annual Meeting.

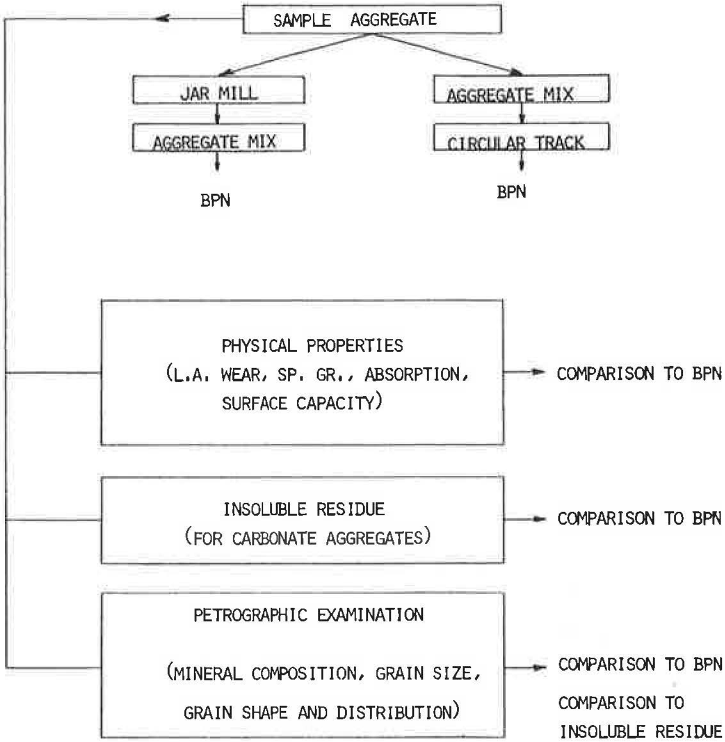

Figure 1. Study procedure.

The major findings of this research involve the hardness of minerals in the various aggregates studied and the mixture of minerals of different hardnesses in the same aggregate. An aggregate containing a mixture of from 50 to 70 percent hard minerals and 30 to 50 percent soft minerals will have a terminal skid resistance after wear that is higher than aggregates containing predominantly soft or hard minerals. Aggregates containing predominantly hard minerals will polish less rapidly than will aggregates containing predominantly soft minerals, but terminal skid-resistance values when reached will be similar. Hardness and percentages of minerals can be determined from petrographic studies of thin sections of aggregates by using the polarizing microscope.

SAMPLE AGGREGATES

Aggregates from 20 sources representative of the range of aggregates used in North Carolina and believed to be fairly representative of those used in other parts of the United States had been previously rated for laboratory skid resistance in tests using the circular track and jar mill methods (1-3). Skid resistance was determined by using the British portable tester in accordance with ASTM Designation E 303-66T(4), and values were recorded as British portable numbers (BPN). Physical properties were obtained by using AASHO and ASTM procedures when applicable (Table 1). Circular track BPN values are given in Table 2.

North Carolina State Highway Commission (NCSHC) No. 13 stone gradation (3) has been used in all of the testing. This is the normal coarse aggregate gradation used in bituminous surface mixes, and the physical properties of this material are given in Table 1. In the jar mill method, aggregate from the No. 13 stone was obtained by sieving the fraction passing the 3/8-in. and retained on the No. 4 sieve.

TABLE 1

PHYSICAL PROPERTIES OF SAMPLE AGGREGATES

Number	General Classification	Grading C Los Angeles Wear Loss[a] (percent)	Bulk Specific Gravity[a]	Water Absorption[a] (percent)	Surface Capacity[b] (percent)	Approximate Asphalt Absorption[b] (percent)
LS-1	Limestone	18.0	2.85	0.30	0.6	0.21
LS-2	Limestone	25.0	2.87	0.40	0.9	0.28
LS-3	Limestone	46.0	2.47	3.35	5.5	2.45
LS-4	Limestone	29.0	2.95	0.30	0.7	0.21
GT-1	Granite	36.0	2.79	0.31	0.6	0.20
GT-2	Granite	63.0	2.67	0.42	1.0	0.29
GT-3	Granite	51.0	2.65	0.50	1.0	0.34
GT-4	Granite	41.0	2.66	0.50	1.0	0.34
GN-1	Granite gneiss	29.0	2.67	0.41	0.8	0.28
GN-2	Granite gneiss	52.0	2.68	0.6	1.1	0.41
GN-3	Granite gneiss	48.0	2.71	0.55	1.0	0.36
GL-1	Gravel	42.0	2.64	0.30	0.5	0.20
GL-2	Gravel	43.0	2.78	1.01	2.1	0.68
SL-1	Slate	17.0	2.78	0.32	0.4	0.21
SL-2	Slate	24.0	2.78	0.33	0.5	0.2
RH-1	Rhyolite	27.0	2.67	0.30	0.6	0.21
TR-1	Traprock (diabase)	15.0	2.77	0.30	0.6	0.20
SS-1	Sandstone (arkose)	N.A.	2.66	2.55	5.5	1.70
SP-1	Expanded glass[c]	23.3	2.05	2.4	5.4	1.60
SO-1	Expanded slate[c]	40.0	1.58	3.5	7.2	2.45

[a]Determined by the Materials Laboratory of the North Carolina State Highway Commission.
[b]Obtained by using Hveem's Method (5, p. 56).
[c]Provided by manufacturer except surface capacity.

MEGASCOPIC AND PETROGRAPHIC STUDY

Thin sections of hard specimens were prepared for all of the mineral aggregates given in Table 1 except for one gravel, GL-1, and the synthetic aggregates, SP-1 and SO-1. These aggregates were not included because sufficiently large pieces were not available. Where an aggregate apparently consisted of more than one rock type or color representative "chunks" were obtained, and thin sections were prepared for each of the variations recognized. A detailed petrographic description of each aggregate is given in other publications (1, 2). Photomicrographs of thin sections obtained from 12 aggregates representative of the aggregate samples are shown in Figure 2. The photomicrographs and the data given in Tables 3 and 4 reveal that aggregates in different classifications as well as some aggregates grouped in the same classification may vary significantly in mineral composition percentage or size and shape of grain or both. Variations of this nature become important when mineral properties of these aggregates are related to their skid-resistance properties.

TABLE 2

RELATIVE RATING OF SKID-RESISTANCE CHARACTERISTICS OF SAMPLE AGGREGATES

Aggregate	BPN Adjusted Value After Exposure		Relative Rating
	8 Hours	16 Hours	
SS-1	61.0	58.5	1
SO-1	52.0	51.0	2
SL-2	46.5	46.0	3
SP-1	46.5	45.0	4
SL-1	46.0	45.0	5
GL-2	45.3	45.0	6
GT-2	45.0	44.5	7
GT-3	45.0	43.5	8
GT-4	44.5	43.0	9
GN-1	44.2	43.0	10
GN-2	43.7	43.0	11
GN-3	43.5	42.5	12
GT-1	43.0	42.0	13
RH-1	43.5	41.5	14
TR-1	43.5	40.5	15
GL-1	42.0	40.0	16
LS-4	41.5	40.0	17
LS-3	41.5	40.0	18
[illegible]	[illegible]	[illegible]	[illegible]
LS-1	36.0	35.0	20

INSOLUBLE RESIDUE TEST

Four of the aggregate samples included in this study were classified as limestone (Table 1). In some geographical areas this type of aggregate has a reputation for polishing readily under the action of traffic, causing pavement surfaces to become slippery after a relatively short period of time.

Some investigators (6-14) have examined limestones from various sources and have found that they are not all alike in their polishing characteristics. These same investigators generally agree that a limestone having some siliceous material in its composition is less likely to polish and become slippery than is one having only carbonate composition. Agreement is not general, however, as to the percentage of siliceous material and the particle size that must be present in a limestone to make it significantly skid resistant.

A method to determine the amount of siliceous material contained in a carbonate aggregate is the acid-insoluble residue test pioneered by Gray and Renninger (11) and

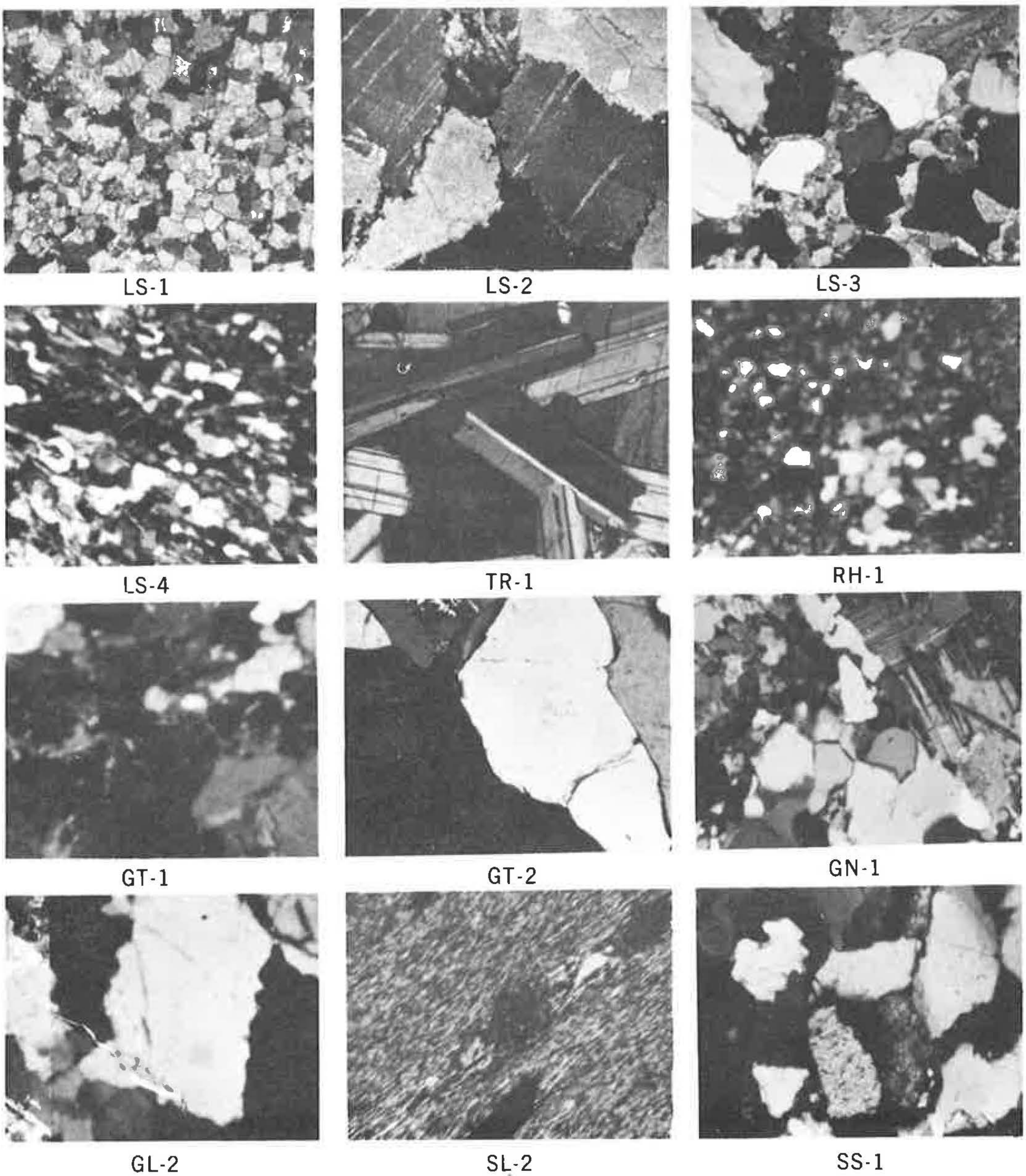

Figure 2. Photomicrographs of 12 selected aggregates under crossed nicols (x35).

modified by others. A modification of the Gray and Renninger method was used to determine the amount of insoluble residue in the four limestone aggregates used in this study.

For each of the four limestone aggregates, three acid-insoluble residue tests were performed by using NCSHC No. 13 stone, and three tests were performed by using only the fraction of material passing the ³⁄₈-in. and retained on the No. 4 sieve. Average gradations indicating percentage and size of residue particles obtained as a result of these tests on each aggregate are given in Table 5.

Some correlation was found between the amount and particle size of insoluble residue and skid-resistance properties of pavement mixes made from carbonate aggregates. Generally, it was found that the higher the amount of insoluble residue was, the higher the skid resistance was.

TABLE 3

MOHS' HARDNESS OF MINERALS IN SAMPLE AGGREGATES

Mineral No.	Mineral Name	Hardness Range
M-1	Chlorite (sericite or kaolinite)	2 to 2.5
M-2	Mica (biotite or muscovite)	2 to 3
M-3	Calcite	3
M-4	Dolomite	3.5 to 4
M-5	Pyroxene (augite)	5 to 6
M-6	Feldspar (plagioclase or orthoclase)	6
M-7	Hematite or magnetite	6
M-8	Olivine	6.5 to 7
M-9	Quartz	7
M-10	Others (apatite, amphilbole, pyrite, epidote, and zircon)	5 to 7.5

ANALYSIS OF FACTORS AFFECTING POLISHING OF AGGREGATES

Skid Resistance and Physical Properties of Aggregates

A comparison between the physical properties of the 20 sample aggregates given in Table 1 and the skid-resistance characteristics after exposure of the same aggregates given in Table 2 has failed to establish a consistent general relationship between any of

TABLE 4

PERCENTAGE OF MINERAL COMPOSITION OF SAMPLE AGGREGATES

Aggregate	M-1	M-2	M-3	M-4	M-5	M-6	M-7	M-8	M-9	M-10
LS-1			93						5	2
LS-2			65	30					5	
LS-3			50	20			5		25	5
LS-4		5	25	30		5			30	5
GT-1 (light)		10				52[a]			35	3
GT-1 (dark)		12				55[a]			20	13
GT-2 (light)		10				65			20	5
GT-2 (dark		10				70			20	trace
GT-3		10				50			35	5
GT-4	2	8				55			35	
GN-1		10				40			40	10
GN-2	4	10				55			25	6
GN-3		15				60[a]			15	10
GL-1									98	2
GL-2 (granite)	2	10				32			50	6
GL-2 (gneiss)		5			30	32			25	8
SL-1	60					20			20	
SL-2	55					20			15	10
RH-1		5				40	15		40	
TR-1					40	50	5	5		
SS-1						50[a]	10		40	

Note: Percentage of mineral composition is approximate. Names and hardness of minerals are given in Table 3. Aggregate identification and more details are included elsewhere (1, 2).

[a]Chemically altered, in part.

TABLE 5

INSOLUBLE RESIDUE OF FOUR LIMESTONES

Sieve	Accumulative Percent Retained[a]							
	LS-1		LS-2		LS-3		LS-4	
	No. 13 Stone[b]	No. 4 Fraction[c]	No. 13 Stone[b]	No. 4 Fraction[c]	No. 13 Stone[b]	No. 4 Fraction[c]	No. 13 Stone[b]	No. 4 Fraction[c]
$^3/_8$ in.	–	–	–	–	–	–	–	–
No. 4	1.3	1.8	0.2	0.7	0.1	0	8.2	22.5
No. 8	2.0	1.9	0.8	1.0	0.1	1	15.8	24.8
No. 16	2.1	1.9	0.9	1.0	0.2	0.1	17.7	25.2
No. 30	2.2	2.3	1.1	1.1	0.7	0.6	17.8	25.3
No. 50	2.2	2.3	1.1	1.2	5.1	6.4	17.8	25.4
No. 100	2.2	2.3	1.3	1.2	23.6	24.6	17.8	25.4
No. 200	2.2	2.4	1.4	1.3	27.7	31.0	17.8	25.4
No. 270	–	2.4	–	1.3	–	31.4	–	25.6
Filter paper (total residue)	7.3	6.7	3.4	33.3	34.1	34.1	44.4	45.4

[a]Each value is average of three tests for 500-gram initial sample.
[b]Gradation given in another paper (3).
[c]Passing $^3/_8$ in. sieve and retained on sieve No. 4.

the physical properties of an aggregate and its skid-resistance characteristics. It had been anticipated intuitively that the rougher the unworn surface texture of an aggregate was, as may be reflected by its absorption or surface capacity and/or specific gravity, the higher its resistance to skidding would be after wear and polishing. Comparisons have revealed that, although this concept may be true for SO-1, SP-1, SS-1, and GL-2 with high absorption, high surface capacity, and high skid resistance, it does not hold true for either LS-3 with high absorption and relatively low skid resistance or, in a reverse manner, SL-1, SL-2, and GT-1 with low absorption and surface capacity and relatively high skid resistance.

A comparison of both Los Angeles and jar mill wear losses of an aggregate to the skid resistance of that aggregate produced no results that could be considered consistent or general. For example, in the case of the four granite aggregates (GT-1 through GT-4), the higher the Los Angeles wear loss was, the higher the skid resistance was; this trend was almost reversed in the case of the limestone aggregates (LS-1 through LS-4).

Skid Resistance of Carbonate Aggregates

All four of the limestone aggregates fall at the lower end of the group of aggregates with respect to skid resistance after exposure (Table 2). LS-3 and LS-4 were about as skid resistant as the lowest of the other aggregates, LS-2 and LS-1 ranged downward, and LS-1 exhibited the lowest skid resistance value of all aggregates included in the research.

Review of the literature on skid-resistance characteristics of limestone aggregates had pointed to the significance of the acid-insoluble residue content of an aggregate in improving the skid-resistance characteristics of that aggregate. By using the polarizing microscope method, we found that the sand-sized (-0.05 mm) insoluble residue that remained from reacting the limestone with hydrochloric acid consisted of hard siliceous particles, mostly quartz. Similar findings have been reported by other investigators (7, 8, 9, 10, 11, 12, 13). Particles smaller than the size of sand included siliceous material and other clay-sized material that was not identified.

An examination of the insoluble residue test data given in Table 5 generally confirms the findings of other investigators that the higher the insoluble residue of a carbonate aggregate is, the better its skid-resistance performance will be. This general statement, however, seems to require some qualification even in the case of this admittedly limited investigation of limestone aggregates.

Low Insoluble Residue Limestones

A comparison of LS-1 and LS-2 reveals that although LS-1 had a more total insoluble residue and more sand-sized residue than did LS-2, the latter showed a significantly higher skid resistance after exposure. Other differences must exist that obscure the differences in percentage of residue.

LS-1 is identified from the petrographic study as a uniform fine-grained limestone composed of about 95 percent subangular calcite (Fig. 2). The presence of about 2.4 percent sand-sized insoluble residue seems to have contributed little to improving skid resistance, and about 5 percent of minus No. 200 residue containing some clayey material may actually have contributed to lower skid resistance. Under exposure, LS-1 polished rapidly to a relatively low-skid-resistance value.

LS-2 contained less sand-sized insoluble residue and less total insoluble residue than did LS-1. An examination of the LS-2 photomicrograph shown in Figure 2 reveals that this aggregate has subangular medium-to-coarse carbonate grains. These grains were estimated to be one-third dolomite (H = 3.5 to 4) or magnesite (H = 3.5 to 5) and two-thirds calcite (H = 3). It is believed that the presence of coarse-grained dolomite or magnesite with slightly higher Mohs' hardness than the calcite ground mass contributed to the improvement of skid-resistance characteristics of LS-2 over LS-1 through differential wear. This belief seems to be reinforced from an examination of worn aggregate particles under a stereoscopic microscope. After wear to terminal polish, LS-2 exhibited greater surface asperity size and roughness than did LS-1.

A more general statement would be that, when the presence of sand-sized insoluble residue is insufficient to influence skid-resistance properties significantly, the presence of any other mineral harder than calcite in a significant percentage may improve the skid-resistance properties of the limestone. Shupe and Lounsbury (8) have reported, in essence, that the higher the calcite content of a limestone is, the more susceptible it is to polishing.

High Insoluble Residue Limestones

The insoluble reside test results of LS-3 and LS-4 (the higher skid-resistant limestones) indicate that amount and gradation of sand-sized residue and amount of total residue should be considered as factors affecting the skid-resistance properties of an aggregate.

LS-3 and LS-4 are equal in skid-resistance properties (Table 2). Both limestones have relatively high insoluble residue percentages but with considerably different particle-size distribution (Table 5). If total residue alone were the determining factor in improved skid resistance, LS-4 would be superior to LS-3. If sand-sized material alone were the determining factor, LS-3 would be superior to LS-4. Neither condition governs, however, and some other explantion must be sought for lack of difference in performance.

Examination of the photomicrographs of LS-3 and LS-4 reveals differences in grain size and size distribution. LS-3 calcite grains form a fine-grained matrix for the larger grains of dolomite, quartz, and other minerals. LS-4 is uniformly fine grained. These grain sizes and grain distribution properties seem to favor LS-3 in resisting polish. Another consideration may be the effect of insoluble residue that passes the No. 200 sieve. It may be that the passing 200 fraction of insoluble residue containing clay minerals is detrimental to skid resistance, and a high percentage of passing No. 200 also may have affected the skid-resistance properties of LS-4 negatively.

It seems clear, however, that the presence of harder impurities mixed with a calcite ground mass does improve the skid-resistance properties of carbonate rocks in some direct relationship to the amount of impurities. Noncarbonate rocks seem to respond to this same general rule involving a mixture of grains having differential hardness.

It is probable that the impurities wear less rapidly than the calcite, leaving surface relief at all times to aid skid resistance. Examination under a stereoscopic microscope of surfaces exposed to wear tends to support this contention.

NONCARBONATE AGGREGATES

Most of the aggregates studied were noncarbonate in mineral composition, and from those aggregates a range of behavior was observed under the wear and polishing exposures. The ratings given in Table 2 reveal that there was a wide range of response within the granites alone. The traprock (TR-1), composed of hard minerals, was low in the ratings, whereas the sandstone (SS-1), composed of about 50 percent hard and 50 percent soft minerals, showed the most favorable response. The photomicrographs (Fig. 2) show wide variations in mineral composition and in grain size, shape, and distribution. The percentages of mineral composition and mineral hardness of all of these aggregates are given in Tables 3 and 4.

From study of the carbonate rocks, it seems that a mixture of hard and soft minerals results in better skid-resistance properties than do soft minerals alone. One possible extrapolation of the carbonate rock observations would be that all hard minerals would be best. This conjecture seems to be refuted when TR-1, a diabase composed of all hard minerals (H = 5.5 to 7), is considered. TR-1 exhibited the lowest skid resistance of the aggregates except for the limestones and GL-1, a quartz gravel (H = 7). It seems reasonable to conclude that the presence of a preponderance of minerals with a small range of hardness is conducive to lower terminal skid-resistance properties, although the harder aggregates will take a longer time than the softer aggregates to reach terminal polish under exposure.

This conclusion suggests that an aggregate composed of both hard and soft minerals would be expected to produce the most desirable skid-resistance characteristics. This concept was suggested several years ago by Maclean and Shergold (15) in England and confirmed by others (8) in the United States. However, not all aggregates with mixed hardness composition are the same, as will be seen in the ensuing discussion, because of other variables involved. A systematic examination of all aggregates was undertaken to determine percentages of minerals and their hardness (Tables 3 and 4) and other features, such as grain size and distribution, for correlation with skid-resistance properties. Hardness data and BPN values are given in Table 6. Grain size and distribution are discussed in subsequent paragraphs.

The photomicrographs (Fig. 2) of GT-1 and GT-2 granites with different skid-resistance properties reveal approximately the same percentages of hard minerals but markedly different grain size. GT-2 has the larger grain size and exhibits superior skid-resistance properties. Granites characteristically are composed of significant quantities of quartz (H = 7), feldspar (H = 5.5 to 6), and some soft mineral like biotite (H = 2.5 to 3). Feldspar alters chemically to sericite (H = 2 to 2.5), and some alteration is observed for GT-1. Los Angeles wear loss for GT-2 was almost twice that for GT-1, which suggests that GT-2 is less well bound and its surface may tend to remain rougher through loss of grains before polishing has occurred. In general, the characteristics observed seem to favor GT-2 over GT-1 for skid-resistance properties. Aggregate called granitegneiss is closely related to granite both in compositional features and in skid-resistance characteristics. A photomicrograph of GN-1, which was used as a standard aggregate throughout this study, is shown in Figure 2. Although the granite-gneiss aggregates fall in the medium-range

TABLE 6

BPN AND RANGE OF MINERAL CONTENT WITHIN INDICATED HARDNESS OF SAMPLE AGGREGATES

Aggregate	BPN[a]	Mineral Content[b] (percent)	
		H ≤ 2 to 4	H ≥ 5 to 7.5
SS-1	58.5	30 to 40	60 to 70
SO-1	51.0	N.A.	N.A.
SL-2	46.0	50 to 55	45 to 50
SP-1	45.0	N.A.	N.A.
SL-1	45.0	55 to 60	40 to 45
GL-2	45.0	10 to 15	85 to 90
GT-2	44.5	10 to 12	88 to 90
GT-3	43.5	10 to 12	88 to 90
GT-4	43.0	8 to 10	90 to 92
GN-1	43.0	8 to 10	90 to 92
GN-2	43.0	10 to 15	85 to 90
GN-3	42.5	15 to 20	80 to 85
GT-1	42.0	10 to 15	85 to 90
RH-1	41.5	10 to 12	88 to 90
TR-1	40.5	0 to 2	98 to 100
GL-1	40.0	1 to 3	97 to 99
LS-4	40.0	60 to 70	30 to 40
LS-3	40.0	65 to 75	25 to 35
LS-2	39.0	95 to 97	3 to 5
LS-1	35.0	93 to 95	5 to 7

[a]After 16 hours of circular track exposure.
[b]Details are given in Tables 3 and 4.

skid-resistance group, they seem to be generally inferior to the granites. This inferiority may, in general terms, be attributed to the more extensive alterations in their compositional minerals, particularly from feldspar to kaolinite or sericite, and to the lesser availability in their composition of the stable harder mineral like quartz.

RH-1 was identified as a rhyolite. The petrographic analysis for this aggregate showed that it had about the same composition as a granite except for the grain size, which was considerably smaller. As shown in the photomicrograph of RH-1, this type of rock has a ground mass of fine subangular grains that result in a dense, relatively nonporous mass susceptible to uniform wearing and polishing except where some crystal phenocrysts interrupt the grain uniformity and help to improve the skid-resistance characteristics of the aggregate. The skid resistance of the rhyolite aggregate was considerably less than that of an unaltered granite having the same composition. This fact seems to confirm the conclusion that grain size is a factor that contributes to the skid-resistance characteristics of an aggregate.

Two crushed gravels, GL-1 and GL-2, were compared. GL-1, a fall-line gravel, is the least skid-resistant noncarbonate aggregate, is 98 percent quartz, and polishes to a low skid-resistance value. GL-2 is a mountain-stream gravel composed of granite and granitegneiss; its skid-resistance characteristics are typical of those aggregates. The photomicrograph of GL-2 (Fig. 2) shows coarse interlocking grains that probably contribute to skid resistance.

An interesting and rather surprising result was obtained when the slates, particularly SL-2, were tested for skid resistance. It had been anticipated that the flakey particle shape and the fine grain size of this type of aggregate would make it susceptible to the development of a relatively well-polished surface under only moderate traffic action. Skid test results have indicated that this is not the case. SL-2 had the highest skid resistance of any of the natural aggregates in this study that actually have been used in North Carolina pavements (Table 2).

The photomicrograph of SL-2 shows a thin section of this aggregate under crossed nicols of the polarizing microscope. The banded, extremely fine grains are composed of only about 45 percent of the naturally hard minerals, but it has been theorized that slates gain a high degree of hardness of grains through metamorphism (16). It is suspected that this type of aggregate derives its high skid-resistance characteristics from the continuous renewal of the sharp-edged, hard, banded, and brittle surface grains that continuously break and "peel off" under the action of traffic, resulting in little or no accumulated polish.

It may be interesting to mention at this point that the synthetic aggregate SO-1, produced from a slate rock, showed high skid-resistance characteristics. These characteristics have been generally attributed to continuous rough surface renewal and to high contact pressure produced between the grain surface and the rubber because of the high porosity of this aggregate, i.e., voids act as soft areas and vesicule walls function as hard areas.

The sandstone (arkose) aggregate SS-1 that was used in this investigation is an excellent example of the type of aggregate that could provide an ideally long-lasting skid-resistant surface. The sandstone specimens produced the highest skid resistance of the aggregates included in this investigation (Table 2).

The composition of this sandstone, shown in Figure 2, is thought to be the primary factor that gives it skid-resistance characteristics. With grain structure of about 50 percent hard, angular to subangular quartz (H = 7) and hematite (H = 6), and 35 to 40 percent kaolinite (H = 2 to 2.5) resulting from the alteration of feldspar, the soft ground mass wears away relatively fast, exposing the hard grains to provide a sandpaper-like surface. Before the asperities of these hard particles have a sufficient wearing action to cause them to polish, the matrix has been worn down to where it can no longer hold the hard particles, allowing them to be dislodged to expose fresh, unpolished particles. This continuous renewal of the pavement surface is believed to give sandstone highly favorable skid-resistance properties.

Our findings regarding the sandstone aggregate are in agreement with those of some investigators (8, 17, 18) who have experimented with sandstone aggregates in the laboratory or have examined actual pavement field test sections and found them to be highly skid resistant.

An important conclusion that may be drawn from the preceding analysis is that those aggregates that are predominantly composed of one type of mineral, or of different minerals with approximately the same hardness, are more susceptible to polishing and becoming nonskid resistant under sustained traffic action than are aggregates composed of different minerals not equal in hardness. Obviously, the aggregates with the softer mineral composition are polished at a faster rate than those with the harder mineral composition. The analysis has confirmed that the presence in the composition of an aggregate of two or more minerals that have significantly different hardness values contributes considerably to giving the aggregate a sustained high skid

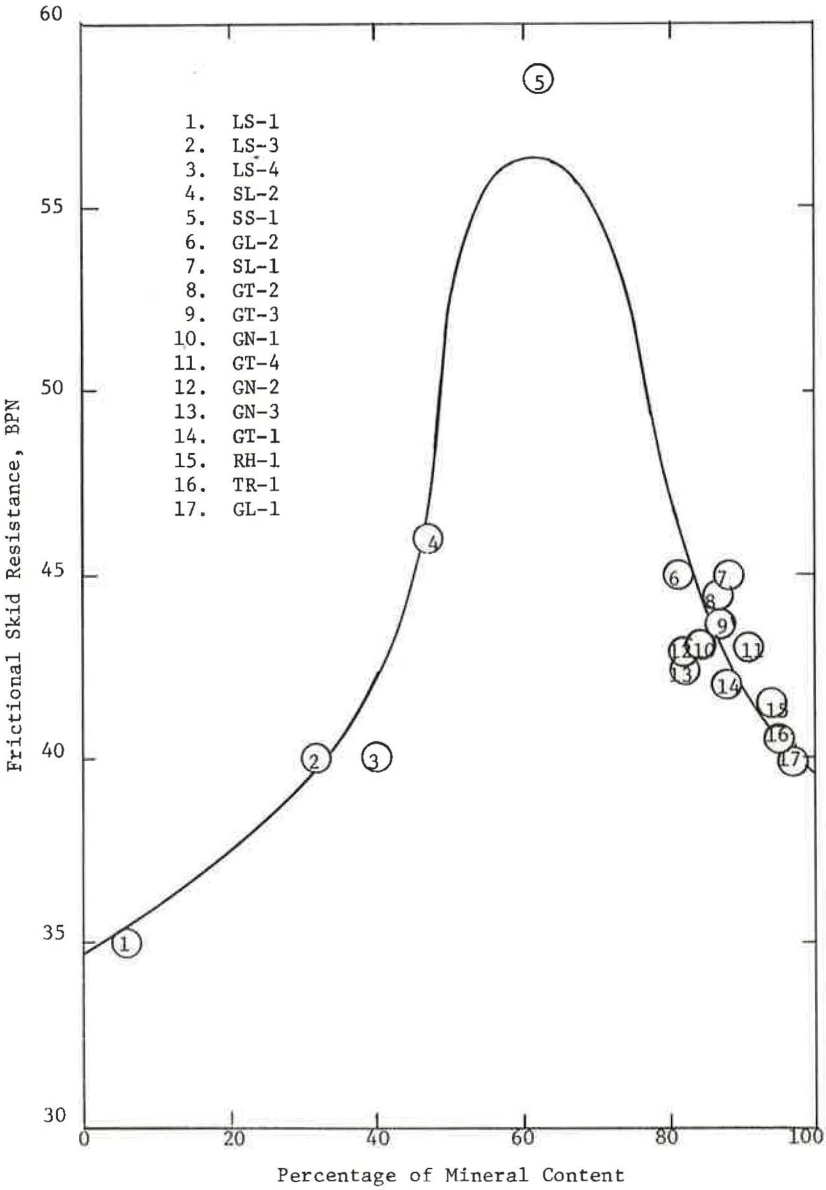

Figure 3. BPN values versus hard mineral content.

resistance under prolonged traffic action; the greater the difference in hardness is, the greater the contribution appears to be. An important observation in this connection is that the proportion of hard to soft minerals appears significant and that there seems to be an optimum proportion that, when exceeded or not attained, results in decreased skid-resistance effectiveness. This concept of optimum proportion of mineral content is shown in Figure 3. The range of values for percentage of mineral content in the sample aggregates is given in Table 6. It is emphasized that the numerical values for mineral content given in Table 6 and shown in Figure 3 are approximate ranges subject to reasonable numerical variation.

Grain size, shape, distribution, and surface capacity seem to contribute significantly to skid-resistance attributes: The larger and the more angular grains, evidently resulting in a coarser and a more textured surface, seem to contribute more effectively to the skid resistance of an aggregate than do grains of smaller size, less angularity, and smoother surface texture. These attributes generally contribute to the modification of the influence of the compositional proportion concept, as was obvious in the case of the fine-grained rhyolite, RH-1, when compared to the coarser-grained granite, although both types of aggregate had similar compositions.

Another factor related to the mineral composition of aggregates in relation to their skid resistance is the distribution of the mineral grains. The more uniform the distribution is of the harder and softer grains, the better the resulting skid resistance is, as in the case of the sandstone aggregate.

SUMMARY AND CONCLUSIONS

The research reported here is part of a broad effort to treat the problem of maintaining an adequate level of pavement skid resistance. This particular portion of the research has been devoted to identification and evaluation of factors that contribute to the skid-resistance properties of aggregates. Previous research has indicated that different aggregates do have different skid-resistance properties and that aggregates may be rated relative to one another through use of wearing and polishing exposures in the laboratory.

The acid-insoluble residue percentages for the four carbonate aggregates examined indicated that skid resistance improved with increased residue and that sand-sized residue probably is more important than total residue.

A general correlation was found between the petrographic properties and the skid resistance of any given aggregate: Aggregates having mixed composition of hard and soft minerals had higher skid resistance than did aggregates consisting predominantly of minerals of the same type or having the same hardness.

The following conclusions seem to be supported by the findings from this research for laboratory skid resistance of aggregates.

1. Pavements made from aggregates composed predominantly of the same mineral or of minerals having a narrow range of hardness, such as most limestones, diabases, and some quartz gravels, will polish to lower levels of skid resistance than will aggregates composed of minerals with a wide range of hardness as measured by Mohs' scale, such as sandstone, granites, gneisses, and slates. Aggregates having softer mineral composition will reach terminal polish levels more rapidly than will aggregates having harder mineral composition.

2. Pavement surfaces made from aggregates having approximately the same mineral composition but with differing grain shape and/or size will produce differing skid-resistance levels. The more angular and the larger are the mineral grains in individual aggregate particles, the higher is the skid resistance of the aggregate particles when incorporated in pavement surfaces.

3. There seems to be an optimum compositional proportion of hard to soft mineral grains in an aggregate for high skid-resistance performance. The optimum seems to fall in the range of proportion of 50 to 70 percent of hard minerals ($H \geq 6$ or 7) to 30 to 50 percent of soft minerals ($H \leq 2$ or 3). The influence of this compositional proportion is apparently modified by the size, shape, and distribution of the mineral grains in the aggregate particles. The larger and the more angular are the hard mineral grains, and

the more uniform their distribution in the softer mineral matrix, the higher is the resulting skid resistance of the aggregate.

4. Generally, carbonate aggregates will polish faster than will other types of aggregates because of the predominance of the soft carbonate minerals in their composition. The higher is the percentage and the harder is the mineral proportion of impurities in a carbonate aggregate, the better will be its skid-resistance performance.

5. The amount of sand-sized insoluble residue, the residue gradation, and the total amount of insoluble residue obtained from a carbonate aggregate should be considered simultaneously in evaluating a carbonate aggregate for skid resistance. The sand-sized portion of insoluble residue seems to have more influence than does total residue on the skid-resistance performance of carbonate aggregates.

6. A long-lasting and highly skid-resistant pavement surface may be obtained by the use of either a natural aggregate or a synthetic aggregate whose sacrificial surfaces are continuously renewed by traffic action.

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