TRANSPORTATION RESEARCH RECORD
523

Formerly issued as Highway Research Record

Skid Resistance

16 reports prepared for the 53rd Annual Meeting
of the Highway Research Board

TRANSPORTATION
RESEARCH BOARD

NATIONAL RESEARCH
COUNCIL

Washington, D. C., 1974

Transportation Research Record 523
Price $6.60
Edited for TRB by Marianne Cox Wilburn

subject areas
26 pavement performance
31 bituminous materials & mixes
33 construction
35 mineral aggregates
40 maintenance, general
51 highway safety
70 legal studies

Transportation Research Board publications are available by ordering directly from the Board. They are also obtainable on a regular basis through organizational or individual supporting membership in the Board; members or library subscribers are eligible for substantial discounts. For further information, write to the Transportation Research Board, National Academy of Sciences, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

These papers report research work of the authors that was done at institutions named by the authors. Tl papers were offered to the Transportation Research Board of the National Research Council for publicatior and are published here in the interest of the dissemination of information from research, one of the major functions of the Transportation Research Board.

Before publication, each paper was reviewed by members of the TRB committee named as its sponsor and accepted as objective, useful, and suitable for publi cation by the National Research Council. The member of the review committee were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the subject coı cerned.

Responsibility for the publication of these reports rests with the sponsoring committee. However, the opinions and conclusions expressed in the reports are those of the individual authors and not necessarily those of the sponsoring committee, the Transportation Research Board, or the National Research Council.

Each report is reviewed and processed according to the procedures established and monitored by the Report Review Committee of the National Academy of Sciences. Distribution of the report is approved by the President of the Academy upon satisfactory completioı of the review process.

The National Research Council is the principal operating agency of the National Academy of Sciences and the National Academy of Engineering, serving government and other organizations. The Transportation Research Board evolved from the 54-year-old Highway Research Board. The TRB incorporates all former HRB activities but also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society.

LIBRARY OF CONGRESS CATALOGING IN PUBLICATION DATA

National Research Council. Highway Research Board.
Skid resistance.

(Transportation research record; 523)

1. Pavements–Congresses. 2. Liability for traffic accidents–Congresses. 3. Motor vehicles–Skidding–Congresses. 4. Motor vehicles–Tires–Congresses. I. National Research Council. Transportation Research Board. II. Title. III. Series: Transportation Research Board; 523.
TE7.H5 no. 523 [TE250] 380.5'08s [625.8] 75-619104
ISBN 0-309-02368-8

CONTENTS

FOREWORD

All but one of the papers included in this RECORD were prepared and presented at the skid resistance symposium and are grouped according to the 4 session topics. These papers will be of interest to those practicing engineers involved in the design, construction, and maintenance of skid-resistant pavement surfaces as well as highway administrative and legal officials.

The first group of papers by Carlson, Gartner, Loutzenheiser, and Somerville and the report of a general discussion focus on the legal liability of highway departments and personal liability of highway officials for injuries resulting from skidding accidents. Included are discussions on claims and recent court decisions involving injuries in wet weather skidding accidents and the problem of defense and settlement of these claims. The papers also include discussions relating to the responsibilities of state and federal highway officials in discharging their legal and moral obligations toward minimizing skidding accidents.

The second group of papers address the problem of measuring and predicting skid resistance properties of materials and surfaces. The paper by Mullen reports on the results of tests to establish a correlation between field wear and laboratory wear. The paper by Mullen, Dahir, and El Madani describes the development and evaluation of 2 laboratory tests for preevaluating aggregates and paving mixtures. Schonfeld's paper describes a surface texture classification system that uses a stereophotographic technique to identify pavement surfaces and correlate the textures with experience and data accumulated from skid-testing devices. The final paper in this group by Van Vuuren, although not a part of the symposium, is included as pertinent information that relates tire inflation pressure to tire contact pressure and, thus, tire-surface skid properties.

The next 6 papers cover the aspects of correcting skid resistance deficiencies by use of bituminous surface treatments. Adam and Shah present an analysis of data collected in Louisiana over a 4-year period on frictional performance of open-graded, plant-mix seal surfaces. Gallaway and Epps discuss the desirability and use of chip seals to improve skid resistance of pavement surfaces, and they provide an evaluation of material properties and design and construction procedures. Seim reports on satisfactory experience with the use of epoxy asphalt surfaces on 3 California toll bridges. Applications included a ¼-in. (6.3-mm) chip seal and a ½-in. (12.7-mm) overlay on 2 old concrete decks and a 1½-in. (38.1-mm) pavement on an orthotropic steel plate deck. Van Til presents a summary of the findings of a National Cooperative Highway Research Program project to identify and evaluate systems intended to improve wear- and skid-resistant pavement surfaces. Havens reports on Kentucky's experience in developing porous sand-asphalt mixtures. These mixes are considered to possess almost all of the skid resistance attributes of other open-graded mixes. Mahone, Hughes, and Maupin outline Virginia's skid resistance experience with dense-graded asphalt concrete mixes.

The last paper is on the topic of wear and texturing of portland cement concrete pavements as they relate to skid resistance. Leyder and Reichert report on 10 years of research and experience in Belgium with the use of deep transverse grooving of fresh portland cement concrete. Included are discussions on techniques and devices for grooving as well as performance data.

A REVIEW OF CASE LAW RELATING TO LIABILITY FOR SKIDDING ACCIDENTS

Robert F. Carlson, California Department of Transportation

One of the greatest problems facing highway departments is slippery pavements in nonfreezing, wet weather. Because of increased legal duties imposed on public entities, highway departments find themselves exposed to liability for accidents that result from what used to be considered purely weather-related causes. Immunities are being weakened, engineering decisions are subject to review, and personal responsibilities are being imposed. A program of skid testing is imperative for early detection of low skid resistance areas. The use of mandatory minimum skid numbers is warned against because of possible adverse legal implications. Grooving, as well as other methods, has proven to be a solution to problems of low skid resistance. Generally, a public entity is not liable for a highway made slippery by rain alone; however, public entities may be held liable for hazardous low skid resistance conditions that result from their own actions or inactions (worn pavements; defectively designed, slippery PCC pavements; unplanted eroding cut-slopes; improperly applied seal coats; and clogged drains and drainage ditches that cause ponding). They also may be found liable when conditions are purely weather created and the hazard is such that the public entity has a duty to remove it entirely or ameliorate it by the use of warning signs. The reasonableness of the public entity's actions generally will be the deciding factor on whether liability will ensue.

•ONE of the most salient legal considerations of skid-resistant qualities of road surfaces is the potential liability arising from an automobile accident caused by the surface of the pavement. Although some accidents may be caused by low skid resistance pavement under dry conditions, the majority of slippery-pavement-related accidents occur during inclement weather. Of course, when snow and ice are on the pavement, it is inevitable that accidents will ensue. These accidents, however, are less related to the skid resistance of the pavement than to the efficiency of highway crews in ameliorating the conditions by means of sand or chemicals. It is primarily under nonfreezing, wet weather conditions that the skid resistance of the pavement surface may be a deciding factor in determining whether a potential accident occurs. Because of this and because public entities now must provide up-to-date skid-resistant highways, many lawsuits are being initiated as a result of what used to be considered purely weather-caused accidents.

As the duty of public entities is enlarged, cases have increasingly involved close questions of fact for the jury. Because of the appellate courts' abhorrence of invading the fact-determining process of the jury, the number of reported decisions does not really reflect the increase in litigation in trial courts. Therefore, I have included cases of personal experience within the California Department of Transportation that were decided or settled at the trial level along with reported decisions to give a better feel of how trial courts and juries are responding to conditions that give rise to accidents and to the potential liability of public entities.

Publication of this paper sponsored by Legal Resources Group Council and Group 2 Council.

SOVEREIGN IMMUNITY

The classical doctrine of "rex non potest pecarre"[1] or "sovereign immunity" was a natural extension of medieval rule by divine right. Obviously, a king who received his power directly from God could not commit tortious acts. Sovereign immunity, which evolved from this archaic justification, remains with us today. To be sure, the present rationale for the perpetuation of a concept that appears to some as outdated[2] lies in contemporary notions of public policy, the necessity of a state's ability to govern, the exercise of police power, and fiscal solvency. For these reasons it is still a general rule that, unless a state has given its consent by way of a constitutional or statutory provision, a state exercising governmental functions,[3] as distinguished from proprietary functions,[4] cannot be made to respond to damages in tort. However, in keeping with the modern trend of courts, providing a remedy to injured parties, several states have enacted statutes that allow a state to be sued in tort under prescribed conditions.[5]

California's experience with sovereign immunity illustrates the present trend of extending liability where possible. In 1961 the California Supreme Court concluded in Muskopf v. Corning Hospital District that "... the rule of governmental immunity from tort liability... must be discarded as mistaken and unjust."[6] After a 2-year moratorium on tort actions against the state, the legislature in 1963 passed the comprehensive California Tort Claims Act.[7] The Act reinstated sovereign immunity except that under certain statutory conditions the state could be sued in tort.[8] One such statu-

---

[1] "The King Can Do No Wrong", 2 Rolle 304.

[2] Muskopf v. Corning Hospital District, 55 Cal.2d 211, 11 Cal. Rptr. 89; Taylor v. New Jersey Highway Authority, 22 N.J. 454, 126 A.2d 313.

[3] Turner v. Superior Court in and for Pima County, 3 Ariz. App. 414, 415 P.2d 129; Faber v. State, 143 Colo. 240, 353 P.2d 609; Cournoyer v. Dayton, 20 Conn. Supp. 48, 122 A.2d 30; Hubbard v. State, 163 N.W.2d 904 (Iowa); Daniels v. Kansas Highway Patrol, 206 Kan. 710, 482 P.2d 46; V.T.C. Lines, Inc. v. City of Harlan, 313 S.W.2d 573 (Kentucky); Vujnovich v. State Through Department of Public Works, 184 So.2d 618 (La.); Sullivan v. Com., 335 Mass. 619, 142 N.E.2d 347; McCree and Co. v. State, 253 Minn. 295, 91 N.W.2d 713; City of Three Forks v. State Highway Commission, 156 Mont. 392, 480 P.2d 826; Opinion of the Justices, 101 N.H. 546, 134 A.2d 279; Schuschel v. Volpe, 84 N.J. Super. 391, 202 A.2d 218; Livingston v. Regents of New Mexico College of Agriculture and Mechanics Arts, 64 N.M. 306, 328 P.2d 78; Bellows v. State, 37 A.D.2d 342, 325 N.Y.S.2d 225; Wrape v. North Carolina State Highway Commission, 263 N.C. 499, 139 S.E.2d 570; Rector v. State, 495 P.2d 826 (Okla.); Moseley v. South Carolina Highway Department, 236 S.C. 499, 115 S.E.2d 172; State v. Clements, Civ. App., 319 S.W.2d 450 (Tex.); Emery v. State, 26 Utah2d 1, 483 P.2d 1296; Kellam v. School Board of City of Norfolk, 202 Va. 252, 117 S.E.2d 96; Creelman v. Svenning, 67 Wash.2d 882, 410 P.2d 606; Forseth v. Sweet, 38 Wis.2d 676, 158 N.W.2d 370.

[4] Smith v. State, 93 Idaho 795, 473 P.2d 937; Carroll v. Kittle, 203 Kan. 841, 457 P.2d 21; Youngstown Mines Corp. v. Prout, 266 Minn. 450, 124 N.W.2d 328; Stadler v. Curtis Gas, Inc., 182 Neb.6, 151 N.W.2d 915; Tiernan v. Missouri New York World's Fair Commission, 48 Misc.2d 376 264 N.Y.S.2d 834; McKenna v. State, 207 Misc. 1008, 141 N.Y.S.2d 809.

[5] Alaska Stat. § 44.80.010 et seq. (Michie Co. 1967); Cal. Gov. Code § 810 (West 1966); Colo. Rev. Stat. § 130-10-1 et seq. (Bradford-Robinson 1963); Conn. Gen. Stat. Ann. § 134-144 (West Additional Supp. 1974); Code of Ga. Ann. § 95-1710 (Harrison Co. 1972); Hawaii Rev. Stat. § 662-1 et seq. (1968); Iowa Code Ann. § 25 A.1 et seq. (West Additional Supp. 1974); Kansas Stat. Ann. § 46-901 (1973), § 68-419 (1972); Ky. Rev. Stat. § 44.070 et seq. (Bobbs-Merrill 1971); La. Rev. Stat. Ann. § 13.5101 et seq. (West 1968); Me. Rev. Stat. Ann. 23 § 457, 23 § 1451, 23 § 3651 (West 1964); Ann. Laws of Mass. ch. 81, § 18 (Michie Co. 1971), ch. 84, § 15 (Michie Co. 1967); Mich. Stat. Ann. § 5.1806 et seq. (Callaghan and Co. Additional Supp. 1974); Nev. Rev. Stat. § 41.031 et seq. (1973); N.Y. Ct. Cl. Act § 8 et seq. (McKinney 1963); Gen. Stat. of N.C. § 143-291 et seq. (Michie Co. 1974); Ore. Rev. Stat. § 30.260 et seq. (1973); Code of Laws of S.C. § 33-229 et seq. (Michie Co. Additional Supp. 1974); Tenn. Code Ann. § 9-801 et seq. (Bobbs-Merrill Additional Supp. 1974); Vernon's Civ. Stat. of Tex. Ann. art. 6252-19 et seq. (Additional Supp. 1974); Utah Code Ann. § 63-30-8 et seq. (Allen Smith Co. Additional Supp. 1973); Vt. Stat. Ann. 19 § 931 et seq. (Equity Publishing Co. Additional Supp. 1974); Rev. Code of Wash. Ann. § 4.92.010 et seq. (Bancroft Whitney-West Additional Supp. 1974).

[6] 55 Cal.2d 211, 213; 11 Cal. Rptr. 89, 90.

[7] Cal. Gov. Code § 810 et seq.

[8] Cal. Gov. Code § 815.

tory condition and a primary basis of liability is for a dangerous condition of public property.[9] The legislature also enacted several immunities that provide exceptions to the statutory basis of liability.[10]

Until recently 1 of the most frequently used defenses that protected public entities from liability for a dangerous condition was "design immunity."[11] Neither a public entity nor its employees are liable for injury caused by the design of construction on public property when that design has been approved in advance by the legislative body of the public entity or by some other body or employee exercising discretionary authority to give such approval. Moreover, when a design is prepared in conformity with standards previously so approved, the public entity is similarly immune. Thus, when a plaintiff alleges a dangerous condition on the highway, the public entity is immune if it can show that the design that caused the condition had been approved by expert highway engineers exercising their discretionary authority.

On January 3, 1972, in Baldwin v. State,[12] the California Supreme Court emasculated the design immunity statute. It held that the design immunity created by California Government Code § 830.6 does not survive when the design, although reasonably approved in advance as being safe, proves in actual operation under changed physical conditions to be dangerous and causes injury. In another case, the court went further to hold that design immunity does not immunize a public entity for its concurrent negligence in failing to warn of a dangerous condition of public property,[13] nor does it apply when a different use than that that was contemplated rather than changed physical conditions makes the public property dangerous.[14] The net result is that juries can, in California, second-guess engineers on their discretionary decisions even though the decisions might have been reasonable at the time they were made.

These same engineers, along with administrators, directors, and highway personnel down to the person with a shovel, are finding themselves subject to exposure to personal liability. Although the cloak of sovereign immunity may protect these people, that cloak, as illustrated, is tattered and full of holes.

The crux of any determination of personal liability is whether the decision or act involves a discretionary function or a ministerial function. Discretionary functions generally require a nondelegable exercise of reason in deciding whether an act will be done and the means with which it will be accomplished.[15] Ministerial functions occur when there is no question of whether the act will be done, and the means of accomplishment are directed, not discretionary.

Thus, it seems obvious that, in addition to the liability that the public will incur, a maintenance worker who fails to sand a highway after being directed to do so will incur personal liability if any accident occurs as a result of that failure. What is not so obvious is the situation where an administrator directs maintenance personnel to sand when necessary to keep the highway safe or to sand only when the temperature drops to 30 F for at least 4 hours. The first directive tends to delegate the discretionary decision of when to sand to the person in the field, thereby enabling application of a possible immunity for discretionary acts. The second directive removes discretion from maintenance personnel, but, because of the obvious potential creation of dangerous untended icy conditions on the highways, the administrator may nevertheless be liable despite the discretionary nature of his or her directive.

The courts have been wrestling with these problems and the dividing line is unclear.[16]

---

[9] Cal. Gov. Code § 835.

[10] Cal. Gov. Code § 830.2-830.8.

[11] Cal. Gov. Code § 830.6.

[12] 6 Cal.3d 424.

[13] Cameron v. State of California, 7 Cal.3d 318.

[14] Davis v. Cordova Recreation and Park District, 24 Cal. Ct. App.3d 789.

[15] Smith v. Cooper, 475 P.2d 78 (Ore.).

[16] Dalehite v. U.S., 346 U.S. 15, 73 Super. Ct. 956; Johnson v. State of California, 69 C.2d 782, 447 P.2d 352 (Cal.); County of Sacramento v. Superior Court, 8 C.3d 479 (Cal.); Sisley v. U.S., 202 F.Supp. 273 (D. Alaska).

But, an unmistakable caveat to engineers and administrators who may be called upon to justify their decisions at some later date is presented.

SKID RESISTANCE

With this in mind, let us turn to the slippery road problem. A study in West Virginia determined that the average rate of wet pavement accidents was 2.2 times the rate of dry pavement accidents, and the upper range was 85 times the dry pavement rate (90, p. 210). Where the wet pavement rate was 85 times the dry pavement rate, the accident cost per vehicle-mile was 50 times the average. On West Virginia Interstate highways, 40 percent of all accidents are wet pavement accidents.

The problem, however, is not limited to West Virginia; its effects are nationwide. In a study conducted by the Highway Research Board in 1969, the problem of slippery pavements was of major concern to 46 percent of the states and of moderate concern to 50 percent. Only 2 states felt it was of minor importance (15, p. 15). In July 1971, the study was updated and showed that only 33 percent of the states considered slippery pavements to be of major concern, and 54 percent considered the problem to be of moderate concern. Six states felt the problem was minor (30, Appendix A). This shows that, although slippery pavements are still a major problem, the picture is improving.

Generally, a wet pavement accident occurs when the coefficient of friction between the pavement and the tires on a motor vehicle is insufficient, manifesting itself in a loss of traction control. The critical factor in such an accident is the contact area of the tire with the wet pavement. This area can be divided into 3 parts. The first is the forward contact area where the tire first meets the pavement and is actually floating on a film of water. This film thickness progressively decreases from forward to back as the individual tread elements traverse the surface and attempt to squeeze out the water between the rubber and the pavement. The second is the transition or draping area. Here, the tire elements penetrate the water film and begin to drape over the greater asperities of the surface of the pavement and make contact with the lesser asperities. The third part is the dry contact portion where tractive effort is developed (7, pp. 2-3).

If the tire does not have sufficient time to squeeze enough water from the pavement asperities to come in contact with the tire, then the entire length is in the condition of forward contact area and the tire actually is floating on a film of water, or hydroplaning. Reduction of this area of squeezing and initial water film penetration is influenced by water viscosity, speed of the vehicle, tire inflation pressure, geometry and depth of tire tread, wind, initial water film thickness, and pavement surface texture. The area of the transition zone depends on both pavement surface texture and dynamic rubber properties. It is, of course, the responsibility of the highway engineer to provide pavements with the surface texture necessary to reduce the areas of squeezing and initial water film penetration, thereby providing good skid resistance.

To diagnose low skid resistance areas, highway departments must undertake some program of skid testing. California has developed its own portable field skid tester that operates on the principle of spinning a rubber-tired wheel while it is off the ground, lowering it to the pavement, and noting the distance it travels against the resistance of a spring before it stops. The device is attached to the rear of a truck that is stationary during the test (89, pp. 40-48). Pennsylvania State University has developed a hand-carried device using a rubber "shoe" that slides along the pavement as the operator pushes the tester. The frictional resistance experienced by the shoe is converted to hydraulic pressure and displayed on a gauge (91). A laboratory tester that also can be used in the field has been developed by the British Transport and Road Research Laboratory. It consists of a pendulum to which a spring-loaded rubber shoe is attached. When the pendulum drops, the shoe is made to slide over the surface to be tested. The attenuation of the rebound serves as a measurement of the friction (92).

The most widely used method of skid resistance is the locked-wheel method. According to ASTM Method E 274, a test tire is installed on the wheel or wheels of a 1- or 2-wheel trailer. For measurement, the trailer is towed at a speed of 40 mph over dry pavement and water is applied in front of the test wheel. The test wheel is locked by a brake. After the wheel has been sliding along the pavement for a certain distance to permit the temperature in the contact patch to stabilize, the force that the friction of the tire contact patch produces, or the resulting torque on the test wheel, is measured and recorded for a specific length of time. The result is reported as a skid number (SN) (30, p. 14).

There are countless methods of measuring skid resistance, the reliability of which vary. It is difficult, if not impossible, to correlate all of them. What is important is to develop a standard for the type of tester that is being used so that relative skid resistance measurements can be used to diagnose potentially troublesome locations.

The Federal Highway Administration has been actively encouraging states to have programs to improve areas of low skid resistance (94, 95, 96, 97, 98). To this end many states have started accumulating statewide skid resistance inventories. In addition, some have called for the establishment of a minimum SN (48, 55). Although this may be desirable from an engineering standpoint in terms of a guideline for scheduling remedial action, it would nevertheless have very serious legal implications as a possible legal standard of care. For example, if a minimum SN of 35 were adopted, and a given stretch of highway had a lower SN but a good accident history, the failure of the public entity to comply with the minimum standard could be used against it in court to prove that the highway is dangerous. This SN is for maintenance purposes only, not for design. Many states already have minimum design SNs specified in their construction contracts. Typically the contract calls for spot testing of the highway to determine compliance with these contract provisions before the highway is ever opened to traffic. The SN itself, rather than the actual safety history of the highway, becomes the criterion.

It is conceded by even the most adamant supporters of the establishment of minimum SNs that many states do not have the funds or manpower, even with federal aid, to bring all their highways up to par immediately. In the interim, these same states would face a further drain on their treasury in the form of judgments to plaintiffs who are able to convince juries that the minimum SN is a legal standard of care, and that noncompliance with it establishes a prima facie dangerous condition of the highway. Determining whether a highway is dangerous is not simple. It cannot be said arbitrarily that a location with a SN of 34 is dangerous and that one with a SN of 36 is not.

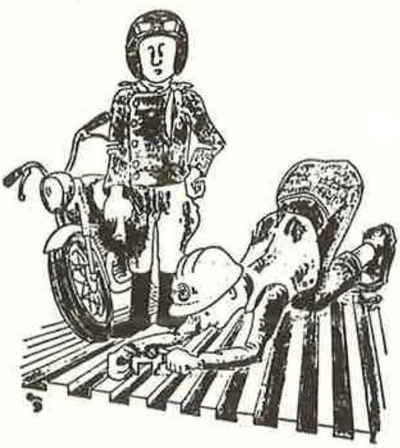
Locations of low skid resistance can be caused at the outset by defective design and construction or through time by deterioration. The circumstances in the former situation arose in the past at a time when highways were designed for a smooth surface to give riding comfort. More recently, the design defect is more one of construction in which the contractor fails to follow the skid resistance specifications in the contract. Pavement deterioration, on the other hand, usually occurs through wear (loss of material), polishing (smoothing of surface microtexture), or changes in the pavement surface structure (binder migration and movement of aggregate particles).

Generally, improvement of low skid resistance locations can be achieved by modifying existing pavement or by applying a new surface. Both approaches are simply means to roughen pavements to increase skid resistance.

Although acid etching and heater planing have been used with varying degrees of success on concrete and bituminous pavements respectively, grooving has proved to be, in most cases, the best solution to the problem of wet weather accidents.

Water runoff is facilitated and the pavement surface is roughened by grooves cut into the pavement with a diamond saw disk. The result is a significant decrease in accidents due to hydroplaning, wet weather skidding, and skidding on horizontal curves (8, 22). The grooves may run longitudinally, transversely, or diagonally. Longitudinal cuts help tracking on curves and transverse cuts, although they are more expensive, are better on straight sections, and facilitate drainage. Diagonal cuts are not widely used because they may cause a vehicle to drift in the direction of the cuts. The cut of the grooves themselves may be V-shaped or rectangular and may vary in depth from 0.062 to ¼ in., in width from ⅛ to ¼ in., and in spacing from ½ to 1 in. Tests in California have shown that a longitudinal, rectangular pattern that is ⅛ in. deep, 0.095 in. wide, and that has ¾-in. spacing is most acceptable to all types of vehicles. A greater width of the grooves may cause motorcycles and light cars to track—a phenomenon resembling being caught in streetcar tracks (1, 88) (Fig. 1). Overall, however, tests of motorcycle

Figure 1. Grooves that are too wide may cause motorcycles to track.



ridability on different types of pavement grooving have not produced any particular pattern that could be considered hazardous (70).

Sometimes, because of the degree of deterioration of the existing pavement, it is necessary to apply a new surface. This can be done by either the plant-mix method or by applying a seal coat in the field. The plant-mix method, which uses maximum polish-resistant aggregate of $^3/_8$ or $^1/_2$ in., provides greater uniformity and thus greater dependability in resurfacing. The direct application of a seal coat in the field, although a widely used method, has inherent potential problems of incorrect bituminous mixture, aggregate proportion, and improper curing, which cause binder migration (bleeding) and loss of aggregate material. Defective applications of seal coats have been a contributing factor in many wet weather accidents.

As is reflected by the emphasis of this discussion, highway skid resistance and what public entities do to improve it bear a direct relationship to the occurrance of wet weather accidents. Without accidents there would be no need to worry about what courts and juries are doing about liability; however, because utopia is far away, it becomes necessary to examine what happens when such accidents do occur, at least as they relate to the liability of public entities.

PUBLIC ENTITY LIABILITY

Given the common knowledge that highways are more slippery when wet than when dry, the general rule is that liability cannot be based on the fact that a highway is slippery from rain alone—there must be some defect, obstruction, or reason within the control of the public entity to hold it liable. Nor can negligence be inferred from the fact that a car skidded or that an accident happened.[17]

Thus, in Gambino v. State[18] the trial court found that the state had caused a skidding accident by constructing a vertical and horizontal curve in a manner not normally encountered, by failing to properly ditch the road, by not properly angling the grooving pattern on the pavement, and by failing to place a speed restriction sign. In its reversal the appellate court stated that all the alleged negligence on the part of the state simply did not contribute to the accident. The state was not an insurer; it had a duty to construct and maintain its highways in a reasonably safe condition in accordance with the terrain encountered and traffic conditions reasonably perceived. But, even so, a certain risk was unavoidable. Sudden storms are a part of nature; 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.

In Walker v. County of Coconino[19] a much broader base of liability was stated[20]:

> If a roadway should suddenly and without fault of the governmental body, come by any means into a condition dangerous to travel, the governmental body is liable for damages occasioned thereby if the governmental body fails to act in a reasonably prudent manner under the circumstances.

---

[17] Eckerlin v. State, 184 N.Y.S.2d 778; Wesley v. State, 272 App. Div. 990, 72 N.Y.S.2d 772; Lahr v. Tirrill, 274 N.Y. 112, 8 N.E.2d 298; Galbraith v. Bush, 267 N.Y. 230, 196 N.E. 36.

[18] 28 A.D.2d 629, 280 N.Y.S.2d 91.

[19] 473 P.2d 472.

[20] Id., at page 475.

Although the Walker case involves ice on the road, the principle stated is specifically extended to any danger whether it be rain, ice, snow, or whatever.[21] The key is the reasonableness of the action or failure to act after the condition manifests itself. Whether there was fault on the part of the public entity in creating the condition goes to the issue of notice of the condition and the reasonableness in remedying it.

Whenever a public entity, either through an independent contractor or through its own employees, constructs or maintains a highway defectively, it is almost certain that liability will ensue for any accidents occasioned thereby. Thus, in a classic case, Veit v. State,[22] the public entity was held liable for injuries arising out of an accident caused by a slippery highway. Approximately 1 month before the accident the highway had been resurfaced with a chip seal. Shortly after completion it became apparent that the emulsion had migrated to the surface and aggregate was being thrown off to the side of the road by the wheels of automobiles and trucks. On 3 occasions the highway department had applied more aggregate to the area; each time the stones were brushed to the sides of the road by traffic. Although signs had been ordered, no SLIPPERY WHEN WET signs to warn of the condition had been placed at the scene. Numerous complaints had been referred to the assistant district highway engineer before the accident. On the day of the accident the plaintiffs were driving at 30 mph in a light rain when, for no apparent reason, the car skidded out of control causing injury.

The court held that ordinarily the state would not be liable for conditions due solely to weather. But, when the state in the resurfacing of a highway creates a dangerous condition and has ample notice of that condition, it is the state's duty to at least advise the public of that condition by erecting warning signs and applying such materials as would remedy the condition.[23]

The California Department of Transportation encountered a similarly caused slippery condition that resulted in a lawsuit. The state had notice of the problem area, but this information was not known by the plaintiffs. During a light rain on a straight stretch of highway a semitrailer hit the slippery area, jackknifed out of control, crossed the centerline, and collided with a fence on the opposite side of the highway. No one was hurt. Ten minutes later a second semitrailer hit the same slippery area, jackknifed, and collided with plaintiffs coming from the opposite direction. Within days after the accident the area was disked to increase the skid resistance; however, this did not solve the problem. The old asphalt had to be burned off and a new chip seal put down.

Had the plaintiffs' attorney been diligent, the state's notice of dangerous condition would have been discovered, and liability would have been virtually automatic. As it turned out, the insurance company for the codefendant truck driver put up $65,000.00 and the state only contributed $5,000.00.

Freeport Transport, Inc. v. Commonwealth of Kentucky, Department of Highways,[24] presents a case in which it appears that the highway department should have left well enough alone. For 9 years there had been no accidents on a particular sharp curve; then a spot patching job was undertaken after which 7 accidents occurred over an 8-month period; the last one gave rise to the lawsuit. The problem was that the surface material used in the patching project began to flake off and exposed the primer coat so that the surface became very slippery when wet. A majority of the court felt that 7 accidents in an 8-month period were sufficient constructive notice to the state that a dangerous condition existed at that location and that the plaintiff was entitled to damages. Two dissenting judges believed there was not evidence that the prior accidents occurred under similar conditions. They voiced the state's position[25]:

---

[21] Compare Shaw v. State, 56 Misc.2d 857, 290 N.Y.S.2d 602.

[22] 78 N.Y.S.2d 336.

[23] Id., at page 339, see also Spence v. State, 165 N.Y.S.2d 896; Carthay v. County of Ulster, 168 N.Y.S.2d 715; Citron v. County of Nassau, 48 Misc.2d 928, 268 N.Y.S.2d 909; but compare Coffey v. State, 193 Misc. 1060, 86 N.Y.S.2d 172; 276 App. Div. 1049, affirmed 96 N.Y.S.2d 303; 276 App. Div. 1049, 96 N.Y.S.2d 304; reargument and appeal denied, 277 App. Div. 831, 97 N.Y.S.2d 918.

[24] 408 S.W.2d 193.

[25] Id., at page 195.

> The majority opinion is the most unrealistic decision this court has reached. The effect of the holding is that it has 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 the road up Chicken Gizzard Ridge and over to Possum Trot?

In another wet weather case involving a construction defect, California contributed to what was at the time the largest out-of-court settlement on record.[26] The case received nationwide attention (93). The action arose from an accident that occurred when a vehicle driven by the plaintiff, a 19-year-old college student, hit a slippery area on a freeway, lost control, and plunged down an embankment. Plaintiff's injuries rendered him a quadriplegic.

The highway was brand new; it had been opened to traffic less than 3 weeks before. During the first month of operation approximately 40 similar accidents had occurred in the same general location. Within 3 days after the accident, highway department engineers tested the road surface for skid resistance. It was discovered that the coefficient of friction was well below the minimum specified in the construction contract (0.25) and below what was considered safe. In addition, witnesses observed an oil film on the road caused by seepage from the asphalt shoulders and the expansion dividers in the PCC pavement. Two days after the accident maintenance employees sanded the area to solve the oil problem. Water also was observed draining across the highway in a wide swath. And, because of the number of cars involved in other accidents that had to be pulled out of the median strip, an accumulation of mud also was on the highway. Thus, the liability picture was rather bleak.

As far as damages went, future medical expenses and lost earning capacity were in excess of $1,500,000.00. This did not even take into account damages for pain and suffering. The state ended up contributing $375,000.00 of a $750,000.00 settlement package with the contractor. Although the amount was startling, it was probably much less than a jury verdict would have awarded. The case was a costly lesson for state inspectors and engineers in new construction.

Another California case that was settled in a trial involved a skidding accident that resulted in quadriplegia.[27] The plaintiff alleged an inadequate coefficient of friction of the pavement itself, the presence of mud on the highway, improper drainage that caused hydroplaning, inadequate superelevation, and lack of a median barrier. The accident history showed more accidents than normal for comparable roadways. Plaintiff also contended that had the highway been grooved the increased skid resistance would have prevented the accident. State engineers agreed with this contention. The highway in question was 20 years old, which illustrates the need for highway departments to continually seek out such locations to alleviate problems of low skid resistance. In this case, simply grooving the highway in the area of the accident would have substantially lowered the risk of having a skidding accident.

Another problem that concerns maintenance more than design is oil accumulation on the pavement. Liability is increasing in this area. In an older case, Sheen v. State Highway Commission,[28] a highway had a large accumulation of spilled oil that was dripped and tracked by the trucks of a nearby oil refinery. The condition had existed for a month before plaintiffs' skidding accident. The court held that there were not sufficient facts to show that the highway itself was defective, which thereby relieved the state from liability. In a later case, Stern v. State,[29] in which the highway had been resurfaced a year earlier, the plaintiff skidded on a slick highway in a heavy rain. There was testimony that the slippery condition was caused by oil leaking from cars backed up because of traffic congestion. The highway was usually slippery for a con-

---

[26] Shipley v. State, Contra Costa County Super. Ct. 97198.
[27] Walton v. State, Santa Barbara County Super. Ct. 80988.
[28] 173 Kan. 491, 249 P.2d 934.
[29] 224 N.Y.S.2d 126.

siderable time until the oil film was washed away. This, of course, is a condition that is prevalent on most highways when it first rains. Nevertheless, this court held that the state was negligent in maintaining the highway in a dangerous condition and in failing to warn the traveling public, until the condition was corrected, that the road was abnormally slippery when wet.[30]

Sometimes it is asserted that the public entity commits an act that, although intended to be beneficial, causes the highway to become more slippery. This happened in California in Catarino v. State,[31] a case in which highway department crews had sprayed the pavement on a bridge with a salt solution to prevent ice from forming. The bridge was posted with signs warning that the bridge was slippery when wet or frosty. The spraying was done at noon; 4 hours later at the time of the accident, the bridge still appeared wet. Investigating officers testified that the bridge was so slippery they had to hang on to the bridge railings to keep from falling down. The plaintiff contended that the spraying caused the pavement to be abnormally slick and was the cause of the accident. The state's witnesses testified that the spraying was not a causative factor and that their actions were reasonable. As it turned out, state maintenance crews were on their way to sand the bridge when the accident occurred. By a poll of 9 to 3 the jury returned a verdict in favor of the state of California; the reasonableness of the state's action was the determinative factor.

Skid resistance is always significantly lessened when water accumulates on the highway either because of faulty drainage design or improper maintenance. In Hampton v. State Highway Commission,[32] it was contended that inadequate design allowed a $2\frac{1}{2}$-in. accumulation of water for 850 ft on a high-speed freeway, which caused plaintiff's car to hydroplane and go out of control. The state's argument was that because hydroplaning could occur with as little water as $\frac{3}{10}$ in. the defective accumulation of $2\frac{1}{2}$ in. was not the proximate cause of the accident. The state Supreme Court did not agree, holding inter alia that, where the highway was too flat and had too few drains that were designed to be frequently clogged, the highway was in a dangerous condition when water accumulated in the traveled portion of the road whenever there was a rain of any consequence.[33]

A similar hydroplaning accident occurred in Smith v. Commonwealth of Kentucky, Department of Highways,[34] where water 2 in. deep was flowing across a highway. The evidence showed that this condition was attributable to the failure of maintenance crews to clean clogged drainage ditches beside the highway. This dangerous condition, coupled with the failure to warn of it, was sufficient to hold the state liable.[35]

In Brown v. Commonwealth of Kentucky, Department of Highways,[36] maintenance crews were in the process of cleaning drainage ditches when plaintiff's vehicle skidded on a muddy section of highway. The mud was deposited on the road during the cleaning operation. The court held that the plaintiff had been warned of the hazardous condition by the state's flagmen and signs, thereby relieving the state from liability. Had the state not provided warnings, the decision would have, no doubt, been otherwise.[37]

A similar California case, Musil v. State,[38] resulted in a hung jury. The plaintiff's vehicle skidded out of control when it hit a muddy spot on a mountain highway. The mud had been tracked on the highway by state maintenance crews in the process of cleaning up a mud slide. No warning signs or barriers advised motorists of the condition. As

---

[30] Id., at page 132.

[31] Stanislaus County Super. Ct. 97169.

[32] 209 Kan. 565, 498 P.2d 236, rehearing denied July 19, 1972.

[33] See also Grady County, Georgia v. Dickerson (C.A.5 Ga.) 257 F.2d 369, cert. denied 358 U.S. 909, 3L. ed. 2d 230, 79 S.Ct 237; Bench v. State, 32 C.A.2d 342, Cal. Rptr. (1973) (regardless of design state may be held liable for failure to warn of accumulation of water in a curve constituting a dangerous condition).

[34] 468 S.W.2d 790.

[35] See also Leader v. South Carolina Highway Department, 244 S.C. 195, 136 S.E.2d 262.

[36] 397 S.W.2d 163.

[37] I.e., Cable v. Marinette County, Wisconsin, 117 N.W.2d 605; J. M. Dellinger, Inc. v. McMillon, 461 S.W.2d 471 (rehearing denied by Ct. Civ. App. of Texas on December 30, 1970).

[38] Stanislaus County Super. Ct. 116813.

a result of the accident, plaintiff was rendered quadriplegic. After several days of deliberation the jury was hopelessly deadlocked at 6 in favor of the plaintiff and 6 in favor of the state. A mistrial was then declared. Because the state felt that it had presented its best case and the plaintiff did not particularly wish to incur the added expenses of retrying the case, although he was prepared to, the case settled at $150,000.00. From these facts, if the case had been retried, a plaintiff's verdict of over ½ million dollars could easily have resulted.[39]

CONCLUSION

As the foregoing discussion illustrates, most cases of potential liability of public entities are determined on the facts of the particular case in a gray area where hard and fast legal rules are difficult to formulate. Public entities may be held liable for hazardous low skid resistance conditions that result from their own actions or inactions, such as worn pavements; defectively designed, slippery PCC pavement; unplanted, eroding cut-slopes; improperly applied seal coats; and clogged drains and drainage ditches that cause ponding. They may also be found liable when conditions are purely weather created and the hazard is such that the public entity has a duty to remove it entirely or ameliorate it by putting up warning signs. The reasonableness of the public entity's actions will generally be the deciding factor on whether liability will ensue.

Thus, all states must have effective and high-priority programs to improve areas of low skid resistance. The money spent for such programs will, in the final analysis, save public agencies millions of dollars as well as lives. It is now more important than ever to create a safe environment for highway users. The courts have laid the responsibility at our doorstep; it is up to us to use the knowledge gained over the past several decades to meet it.

REFERENCES

1. Apostolos, J. Pavement Grooves vs. Motorcycles. Random Samples, California Dept. of Transportation, Sept.-Oct. 1973, pp. 2-6.
2. Adam, V. A Look at Open Mixes to Improve Skid Resistance. Public Works, July 1971, pp. 48-51.
3. Aquaplaning and High-Speed Wet Grip. Serial No. 1190, Nov. 1964, 9 pp.
4. Baca, G. S. Legal Aspects of Skid-Resistant Surfaces—A California Viewpoint. Proc. FHWA Conf. Skid Resistance Surface Courses, April 1972, 7 pp.
5. Baca, G. S., and Gowan, B. C. Legal Liability of Local Government for Negligence in Construction and Maintenance on Public Rights of Way. California Dept. of Transportation, 1972, 8 pp.
6. Baldwin, D. M. Assembly and Use of Accident Data. Highway Research Record 376, 1971, pp. 14-17.
7. Beaton, J. L. Skidding on Highways, Preventive and Curative Measures. Civil Engineering, ASCE, 1970.
8. Beaton, J. L., Zube, E., and Skog, J. Reduction of Accidents by Pavement Grooving. HRB Spec. Rept. 101, 1969, pp. 110-125.
9. Brach, R. M. Friction and the Mechanics of Skidding Automobiles. Highway Research Record 376, 1971, pp. 99-106.
10. Brewer, R. A. Epoxy-Asphalt Open-Graded Pavement as a Skid-Resistant Treatment on the San Francisco-Oakland Bay Bridge. HRB Spec. Rept. 116, 1971, pp. 42-47.

---

[39] See also Kenyon v. State, 21 A.D.2d 851, 250 N.Y.S.2d 1007 [accumulation of dirt on highway (brought by construction trucks), which became slippery when it rained]; Welch v. Amalgamated Sugar Company (D. Idaho, S.D.) 154 F.Supp. 3 (accumulation of mud on highway dropped from trucks entering highway from defendant's property); City of Houston v. Hagman, 347 S.W.2d 355 (city held liable for dangerous condition of street covered with mud caused by rain eroding a dirt embankment; city knew of condition for some time and did nothing to correct it or warn of it).

11. Burnett, W. C., Gibson, J. L., Kearney, E. J., and Balmer, G. G. Skid Resistance of Bituminous Surfaces. Highway Research Record 236, 1968, pp. 49-60.
12. Campbell, M. E., and Titus, R. E. Spotting Skid-Prone Sites on West Virginia Highways. Highway Research Record 376, 1971, pp. 85-96.
13. Carr, C. I. Contribution of the Rubber Compound to the Wet Skid Resistance of Tires. Highway Research Record 214, 1968, pp. 1-6.
14. Colley, B. E., Christensen, A. P., and Knowlen, W. J. Factors Affecting Skid Resistance and Safety of Concrete Pavements. HRB Spec. Rept. 101, 1969, pp. 80-99.
15. An Inventory of Existing Practices and Solutions to Slippery Pavements—1969. Committee on Surface Properties—Vehicle Interaction, Highway Research Circular 106, Feb. 1970, 21 pp.
16. Campbell, M. E. The Wet-Pavement Accident Problems: Breaking Through. Traffic Quarterly, Vol. 25, No. 2, April 1971, pp. 209-214.
17. Cross, M. L. Liability of State, Municipality, or Public Agency for Vehicle Accident Occurring Because of Accumulation of Water on Street or Highway. ALR 2d, Vol. 61, pp. 425-440.
18. Dahir, S. H. M., and Mullen, W. G. Factors Influencing Aggregate Skid-Resistance Properties. Highway Research Record 376, 1971, pp. 136-148.
19. Daughaday, H., and Balmer, G. G. A Theoretical Analysis of Hydroplaning Phenomena. Highway Research Record 311, 1970, pp. 1-10.
20. Determining Pavement Skid Resistance Requirements at Intersections and Braking Sites. NCHRP Research Results Digest 50, Nov. 1973, 12 pp.
21. Domandl, H., and Meyer, W. E. Measuring Tire Friction Under Slip With the Penn State Road Friction Tester. Highway Research Record 214, 1968, pp. 34-41.
22. Farnsworth, E. E. Continuing Studies of Pavement Grooving in California. HRB Spec. Rept. 116, 1971, pp. 134-137.
23. Gallaway, B. M., Rose, G. G., Shiller, R. E., Jr., Kao, T. Y., and Hutchinson, J. W. The Relative Effects of Several Factors Affecting Rainwater Depths on Pavement Surfaces. Highway Research Record 396, 1972, pp. 59-71.
24. Glennon, J. C. The Frictional Requirements of Passing Vehicles. Highway Research Record 396, 1972, pp. 22-32.
25. Goodwin, W. A. Pre-Evaluation of Pavement Materials for Skid Resistance—A Review of U.S. Techniques. HRB Spec. Rept. 101, 1969, pp. 69-79.
26. Goodwin, W. A. Safety Inspection of the Highway and the Vehicle. Highway Research Record 376, 1971, pp. 21-28.
27. Hankins, K. D. The Degree of Influence of Certain Factors Pertaining to the Vehicle and the Pavement on Traffic Accidents Under Wet Conditions. Texas Highway Department, Departmental Res. Rept. 133-3F, Sept. 1970, 57 pp.
28. Hankins, K. D., Morgan, R. B., Ashkar, B., and Tutt, P. R. Influence of Vehicle and Pavement Factors on Wet-Pavement Accidents. Highway Research Record 376, 1971, pp. 66-84.
29. Harris, A. J. Road Surface Texture and the Slipperiness of Wet Roads. Highway Research Record 214, 1968, pp. 18-23.
30. Skid Resistance. NCHRP Synthesis of Highway Practice Rept. 14, 1972, 70 pp.
31. Hilgers, H. F. Slick When Wet. Texas Highways, Vol. 10, No. 1, Jan. 1963, pp. 9-11.
32. Horne, W. B. Tire Hydroplaning and Its Effects on Tire Traction. Highway Research Record 214, 1968, pp. 24-33.
33. Howard, G. Beat the Water Wedge. Autocar, Jan. 1964, 3 pp.
34. Humphreys, J. B., and Moore, A. B. High-Speed Skid Testing at University of Tennessee. Highway Research Record 376, 1971, p. 38.
35. Ivey, D. L., Keese, C. J., Neill, A. H., Jr., and Brenner, C. Interaction of Vehicle and Road Surface. Highway Research Record 376, 1971, pp. 40-53.
36. Kearney, E. J., McAlpin, G. W., and Burnett, W. C. Development of Specifications for Skid-Resistant Asphalt Concrete. Highway Research Record 396, 1972, pp. 12-20.
37. Keen, H. M. Design for Safety. Highway Research Record 214, 1968, pp. 7-12.

38. Kubie, W. L., Gast., L. E., and Cowan, J. C. Preliminary Report on Skid Resistance of Linseed Oil-Coated Concrete. Highway Research Record 214, 1968, pp. 42-49.
39. Kummer, H. W., and Meyer, W. E. Tentative Skid-Resistance Requirements for Main Rural Highways. NCHRP Rept. 37, 1967, 87 pp.
40. Liability for Damage or Injury by Skidding Motor Vehicle. ALR, Vol. 58, pp. 264-287.
41. Liability for Damage or Injury by Skidding Motor Vehicle. ALR, Vol. 113, pp. 1002-1055.
42. Locked-Wheel Pavement Skid Tester Correlation and Calibration Techniques. NCHRP Research Results Digest 49, Sept. 1973, 5 pp.
43. MacKenzie, A. J. Spray Grip Anti-Skid Surfacing Materials. Highway Research Record 376, pp. 61-62.
44. Mahone, D. C., and Runkle, S. N. Pavement Friction Needs. Highway Research Record 396, 1972, pp. 1-11.
45. Mahone, D. C., and Shaffer, R. K. Corrective Programs for Improving Skid Resistance. Highway Research Record 376, 1971, pp. 54-58.
46. Marsh, B. W. Summary of Workshop on Anti-Skid Program Management. Highway Research Record 376, 1971, pp. 5-6.
47. Martinez, J. E., Lewis, J. N., and Stoker, A. J. A Study of Variables Associated With Wheel Spin-Down and Hydroplaning. Highway Research Record 396, 1972, pp. 33-44.
48. McCullough, B. F., and Hankins, K. D. Skid Resistance Guidelines for Surface Improvements on Texas Highways. Highway Research Record 131, 1966, pp. 204-217.
49. McCullough, F. Field Evaluation of the Saw Cut Method. Texas Highways, Vol. 10, No. 1, Jan. 1963, pp. 11-12.
50. Meyer, W. E. Friction and Slipperiness. Highway Research Record 214, 1968, pp. 13-17.
51. Meyer, W. E., and Kummer, H. W. Pavement Friction and Temperature Effects. HRB Spec. Rept. 101, 1969, pp. 47-55.
52. Millard, R. S. Anti-Skid Measurement. Highway Research Record 376, 1971, pp. 59-60.
53. Mills, J. A., III. A Skid Resistant Study in Four Western States. HRB Spec. Rept. 101, 1969, pp. 3-17.
54. Moore, D. F. Prediction of Skid-Resistance Gradient and Drainage Characteristics for Pavements. Highway Research Record 131, 1966, pp. 181-203.
55. Moore, D. F. Recommendations for International Minimum Skid-Resistant Standard for Pavements. HRB Spec. Rept. 101, 1969, pp. 35-38.
56. Moore, D. F. The Logical Design of Optimum Skid-Resistant Surfaces. HRB Spec. Rept. 101, 1969, pp. 39-46.
57. Mortimer, T. P., and Ludema, K. C. The Effects of Salts on Road Drying Rates, Tire Friction, and Invisible Wetness. Highway Research Record 396, 1972, pp. 45-58.
58. Moyer, R. A. History of Anti-Skid Program Management in the United States. Highway Research Record 376, 1971, pp. 1-4.
59. Mullen, W. G., Dahir, S. H. M., and Barnes, B. D. Two Laboratory Methods for Evaluating Skid-Resistance Properties of Aggregates. Highway Research Record 376, 1971, pp. 123-135.
60. Nixon, J. F. Status of Skid Management Program in Texas. Highway Research Record 376, 1971, p. 65.
61. Oliver, D. C. A Synthesis of Case Law Jurisprudence Relating to Wet-Weather Highway Conditions. Highway Research Record 376, 1971, pp. 29-38.
62. Oliver, D. C. The Law of Liability as it Relates to Low Skid Resistant Roadways. Proc. FHWA Conf. Skid Resistance Surface Courses, April 1972, 15 pp.
63. Peebles. Legal Aspects of Skid-Resistance Measuring. Proc. FHWA Conf. Skid Resistance Surface Courses, April 1972, 12 pp.

64. Quinn, B. E., Engja, H., and Zable, J. L. Using a Modified Bureau of Public Roads Roughometer to Measure Pavement Roughness Spectra. Highway Research Record 311, 1970, pp. 26-35.
65. Ricker, E. R. Use of Accident Data to Identify Wet-Pavement Locations in Pennsylvania. Highway Research Record 376, 1971, pp. 17-20.
66. Rizenbergs, R. L., and Ward, H. A. Skid Testing With an Automobile. Highway Research Record 189, 1967, pp. 115-136.
67. Rogers, R. V. Municipal, School, and State Tort Liability. American Jurisprudence, 2nd Ed., Vol. 57, pp. 1-271.
68. Sandvig, L. D., MacGregor, L. M., Shaffer, R. K., Martin, J. P., and Minnick, J. Development and Results of a Skid Research and Road Inventory Program in Pennsylvania. HRB Spec. Rept. 101, 1969, pp. 18-34.
69. Schonfeld, R. Photo-Interpretation of Skid Resistance. Highway Research Record 311, 1970, pp. 11-25.
70. Sherman, G. B. Effect of Pavement Grooving on Motorcycle Ridability. California Department of Transportation, Highway Research Rept. M&R 633126-6, Nov. 1969, 28 pp.
71. Sherman, G. B. Grooving Patterns Studies in California. Highway Research Record 376, 1971, pp. 63-64.
72. Sherman, G. B. Skid Tester Correlation Study. California Department of Transportation, Highway Research Rept. M&R 633126-7, Oct. 1971, 43 pp.
73. Sherman, G. B., Matthews, J. A., and Page, B. G. Statewide Skid Resistance Inventory. California Department of Transportation, March 1973, 78 pp.
74. Sherman, H. W. Investigation of Injury-Producing Automobile Accidents at the Highway Research Institute. Highway Research Record 376, 1971, p. 37.
75. Smith, H. A. Locked-Wheel Pavement Skid Tester Correlation and Calibration Techniques and Requirements for Wear-Resistant and Skid-Resistant Highway Pavement Surfaces. AASHO, 1972, 17 pp.
76. Spellman, D. L. Texturing of Concrete Pavement. HRB Spec. Rept. 101, 1969, pp. 100-103.
77. Spellman, D. L., Woodstrom, J. H., Bailey, S. N., and Spring, R. J. Effect of Broom Texture on Motorcycle Ridability. California Department of Transportation, Highway Research Rept. CA-HY-MR 3126-8-72-32, Aug. 1972, 21 pp.
78. Spickelmire, L. S. Construction of Nonskid Pavement Surfaces. HRB Spec. Rept. 101, 1969, pp. 104-109.
79. States. Corpus Juris Secundum, Vol. 81, pp. 850-1348.
80. Stiffler, A. K. Relation Between Wear and Physical Properties of Road Stones. HRB Spec. Rept. 101, 1969, pp. 56-68.
81. Tyres and Skidding. Autocar, Jan. 1959, pp. 6-7.
82. Multidisciplinary Accident Investigation Summaries. U.S. Dept. of Transportation, NHTSA Rept. DOT HS 601, Vol. 4, No. 9, 326 pp.
83. Van Alstyne, A. California Government Tort Liability. California Continuing Education of the Bar, 1964.
84. Van Alstyne, A. California Government Tort Liability Supplement. California Continuing Education of the Bar, 1969.
85. Weller, D. E., and Maynard, D. P. The Influence of Materials and Mix Design on the Skid Resistance Value and Texture Depth of Concrete. RRL Rept. LR334, 1970, 42 pp.
86. Weller, D. E., and Maynard, D. P. The Use of an Accelerated Water Machine to Examine the Skidding Resistance of Concrete Surfaces. RRL Rept. LR333, 1970, 40 pp.
87. Whitehurst, E. A. Problems Associated With Anti-Skid Program Management. Highway Research Record 376, 1971, pp. 7-13.
88. Zube, E. Coefficient of Friction of Various Grooving Patterns on PCC Pavement. California Department of Transportation, Highway Research Rept. M&R 633126, June 1968, 21 pp.
89. Zube, E., and Skog, J. Skid Resistance of Screenings for Seal Coats. Highway Research Record 236, pp. 29-48.

90. Campbell, M. E. The Wet-Pavement Accident Problem: Breaking Through. Traffic Quarterly, Vol. 25, No. 2, April 1971, p. 210.
91. Kummer, H. W., and Meyer, W. E. The Penn State Road Friction Tester as Adapted to Routine Measurement of Pavement Skid Resistance. Highway Research Record 28, 1963, pp. 1-31.
92. Giles, C. G., et al. Development and Performance of the Portable Skid Resistance Tester. ASTM Spec. Tech. Publ. 326, pp. 50-74.
93. Jinx Strip. Newsweek, Sept. 25, 1967.
94. Pavement Skid Resistance. FHWA, Circular Memorandum, April 21, 1967.
95. Construction of Pavement Surfacing to Provide Safer Coefficient of Skid Resistance. FHWA, Instruction Memorandum 21-3-68, April 29, 1968.
96. Plant Mix Seal Coats. FHWA, Circular Memorandum, May 13, 1968.
97. Highway Safety Improvement Program. FHWA, Policy and Procedure Memorandum 21-16, May 3, 1972.
98. Skid Accident Reduction Program. FHWA, Instruction Memorandum 21-2-73, July 19, 1973.

ENGINEERING AND ADMINISTRATIVE CONSIDERATIONS IN CONSTRUCTING, MAINTAINING, AND TESTING SKID-RESISTANT PAVEMENTS

William Gartner, Jr., Florida Department of Transportation

Certain legal considerations, safety obligations, engineering concerns, and administrative problems are identified that must be addressed by the engineer-administrator in relation to constructing and maintaining pavements with adequate skid resistance properties and testing those skid properties. The monetary effect of claims against the state resulting from skidding accidents is of concern. This problem is further complicated by the fact that some Florida courts have recently abolished the concept of contributory negligence and replaced it with the doctrine of comparative negligence, which has complicated the state's responsibilities. The safety obligations of any state highway department include identifying high accident sites, improving maintenance, and providing a clear roadside policy. The engineer-administrator, in considering skid resistance, must be concerned with (a) testing for skid resistance, (b) various design and construction problems involved in providing a skid-resistant pavement, and (c) inability to attain or maintain skid resistance levels that are being advanced as recommended minimums in various publications. The engineer-administrator must also be concerned with the shortage of materials and the increased cost of importation, particularly in light of the energy crisis, and be aware of possible reductions in the frictional needs of traffic.

•CARLSON (1) has discussed the increasing instances of claims brought against highway departments and highway officials for liability because of injuries received in wet weather skidding accidents and has reviewed recent court decisions in this field. Because of the experience in California, the highway administrators in Florida are deeply concerned over the effect of Chapter 73.313 of the Laws of Florida. This Act, in essence, waives the sovereign immunity of the state effective January 1, 1975, for tort claims up to $50,000 for any injury to any 1 person or $100,000 for any 1 incident or occurrence. Amounts in excess of this would require passage of a claims bill.

LEGAL CONSIDERATIONS

During fiscal year July 1, 1969, through June 30, 1970, there was in effect in Florida a temporary waiver of sovereign immunity. As a result of this waiver, approximately 40 lawsuits were filed against the Florida DOT. During that 1-year period, we had 1 attorney busy full time defending those suits. In addition, we have had another attorney assisting on several cases. Since the time of that sovereign immunity waiver, the Supreme Court of Florida has abolished the concept of contributory negligence and replaced it with the doctrine of comparative negligence. Contributory negligence is a common law doctrine in which the negligence of the plaintiff completely bars any

Publication of this paper sponsored by Legal Resources Group Council and Group 2 Council.

recovery by the plaintiff against the defendant. As an example, if a plaintiff should go off the road and become injured as a result of faulty maintenance of the highway by the department, but it can also be shown that he or she was negligent in that he or she had been drinking and thus had impaired reflexes, he or she may not recover under the doctrine of contributory negligence in a suit against the department. Comparative negligence, on the other hand, is a doctrine in which the negligence of the respective parties is apportioned and the damages of each party would then be shared in proportion to the negligence. Thus, in the last example, if the department's fault in negligence were determined to constitute approximately 50 percent of the cause of the accident and the intoxication 50 percent of the cause of the accident, the department would share in 50 percent of injuries to the plaintiff. We expect that, as a result of the doctrine of comparative negligence by the supreme court, the number of cases against the department will increase.

Of utmost concern to administrators is the possible difficulty in obtaining adequate insurance coverage. As a result of the waiver of sovereign immunity, the highway department must consider carrying insurance to cover claims. The advantages of having insurance coverage are that the premium is budgetable, does not add an increased work load on the staff, and permits convenient handling. These advantages must be measured against the cost. Florida's experience with the 1-year waiver of sovereign immunity would indicate that the cost of the department's handling claims, considering its size and exposure, would be considerably less expensive than its carrying liability insurance with proper limits.

Another concern must be the training of the investigating officer. In all too many instances, an investigating officer notes that the vehicle skidded on pavement before the accident and lists "slippery pavement" as a contributory cause of the accident. A pavement can be considered deficient in skid resistance only in relation to the demand of traffic upon it. Skidding will occur on a dry pavement of high skid resistance if a sufficiently violent maneuver (such as braking to a stop from 60 mph in 100 ft or negotiating a 200-ft radius curve at 60 mph) is attempted. Such demands can arise in an emergency or as a result of extremely poor driving, but the resulting skid cannot be blamed on the pavement. It is only when skidding at curves is caused by maneuvers that are within the range of normal demand (accelerating, braking, or cornering by a majority of drivers under normal conditions) or intermediate demand (last-minute braking or steering corrections caused by inattention, misjudgment, or unusual incidents) that pavement skid resistance should be considered inadequate. Neither can skidding caused by normal braking or cornering on a wet pavement with adequate skid resistance be attributed to the pavement if the vehicle is deficient in such respects as bald tires, improper brake adjustment, faulty steering components, poor suspension, and the like.

Another concern of our attorneys in defending the department against liability suits is that all records of the department are open to the public. For this reason, whenever we identify and make a record of a slippery pavement during our statewide inventory of pavement conditions or during our spot hazard investigations, the department is placed in special jeopardy until the problem is corrected.

Attorneys defending the department in liability cases also are concerned about the influence on juries of published recommended minimum skid numbers (SNs). The Federal Highway Administration and most highway departments have been careful not to publish "magic numbers" that indicate poor skid resistance because they recognize that a pavement can be deficient in skid resistance only in relation to the demand of traffic upon it. Several researchers, however, have not felt so restrained. Even though the publications are careful to point out that the relative skid resistance demand on a pavement is the most important factor concerning the minimum skid resistance that should be applied, both NCHRP Report 37 (2) and NCHRP Report 154 (3) have published recommended minimum "magic numbers." NCHRP Report 154 (3) lists its "magic numbers" on the basis of the 99th percentile of the demand of drivers that use a site. Using this criterion, the report recommends a SN_{40} value of 55 for certain driving conditions. Because of the aggregates available in Florida, I am not certain (in fact, I have severe doubts) that this level of skid resistance can be either attained or maintained under high-volume traffic.

SAFETY OBLIGATIONS

It is obvious that highway departments have certain safety obligations. Among these are identifying high accident sites, improving maintenance, and providing a clear roadside policy. Two other areas that the highway departments are going to have to support are improved driver education and vehicle inspection programs.

ENGINEERING CONCERNS

The engineer-administrator, in considering the overall skid resistance problem, must be concerned with (a) testing for skid resistance, (b) various design and construction problems involved in providing a skid-resistant pavement, and (c) the inability to attain or maintain the skid resistance levels that are being advanced as recommended minimums in various publications.

The major engineering concern in testing for skid resistance is the repeatability and reproducibility of the test procedure and equipment. Most highway agencies in the United States measure pavement skid resistance with locked-wheel, pavement skid testers in general conformance with ASTM Method E 274. However, the repeatability of measurements by this type of tester and the correlation between testers of this type are generally not adequate.

A degree of uncertainty must be accepted with any measurement because of imperfections in the measurement process and the variability of the item being measured. NCHRP Report 151 (4) had as its objectives developing and verifying methods for improving the ability to measure reliably the skid resistance of wet pavement surfaces. The approach involved (a) contacts with skid tester owners to collect information on test equipment and operating procedures, (b) conducting laboratory and field experiments to determine the effect of specific variables on skid resistance measurements, (c) computer simulation studies on the influence of equipment dynamics on skid testing performance, (d) developing tentative recommendations for reducing variability in skid resistance measurements, and (e) conducting a 2-week skid tester correlation program to verify and modify the tentative recommendations.

The primary activity that led to the essential findings of the project was the planning and conducting of a skid tester correlation program at Pennsylvania State University from October 2 to 13, 1972. Nine state highway departments, 2 universities, and 1 county provided skid testers and crews for participation in the program. During the first part of the program, each of the 12 testers made 5 skid tests on each of 4 pavement surfaces at speeds of 40, 30, 50, and 40 mph for 80 tests per tester. An average of 5 tests was considered the measured SN for a particular speed, pavement surface, and tester. After completing the first series of tests, a tabulation of the data indicated a standard deviation of 4.08 from the mean SN for all testers in the condition in which they arrived and operated in accordance with their normal procedure. The range of mean SN values for all testers was as high as 24 for a particular surface. After corrective measures including force and load calibrations, installation of standard water nozzles and ASTM E 17 tires from a single production batch, and procedural adjustments were applied, each tester repeated the entire series of tests. The standard deviation from the mean of the second set of data was reduced to 1.24 and further reduced to 1.04 after temperature corrections were applied. Table 1 gives the reduction in standard deviations that resulted from various corrective measures. If results are to be meaningful very careful calibration of both the testers and the procedures must be made.

Highway administrators also must be concerned with varying pavement conditions. Skid resistance of pavement is not an absolute value; it depends on conditions during tests and on the testing method. It particularly depends on the wear characteristics of the aggregate in the pavement and the amount of debris, including rubber accumulation or oil drippings, on the pavement.

In Florida, a fatal accident occurred that appears to have been the result of inadequate cleaning of a completed construction site. The construction involved excavating a portion of the pavement and temporarily storing backfill material on the pavement. When the construction crew completed its work, it cleaned up the construction

Table 1. Reduction in standard deviations with various corrective measures.

Corrective Measure	Standard Deviation
Testers in "as arrived" condition	4.08
Data evaluation procedures corrected	3.25
Uniform watering nozzles and tires mounted	2.83
Force and load calibration	1.53
Correction for zero drift	1.24
Correction for temperature drift	1.04

site, which included shoveling all loose material from the pavement and light brushing with a broom. Nevertheless, a red clay stain was left on the pavement. That night there was a light rain, and a driver who attempted a passing maneuver at that spot went out of control and was fatally injured.

Concerning the design and construction problems in providing a skid-resistant pavement, the engineer must be concerned with the critical nature of the asphalt content, the critical nature of the voids content in the aggregate, the shape factor of the aggregates, and any postconstruction consolidation resulting in wheel-path rutting.

ADMINISTRATIVE CONCERNS

The energy crisis and the general shortage of materials are making it increasingly difficult to meet the demands of highway construction, particularly those related to providing a skid-resistant surface. In Florida, as in many other parts of the United States, naturally occurring aggregates are primarily limestone. Aggregates with suitable skid resistance must be imported. We are currently importing slag material and other aggregates from as far away as Tennessee. The administrator must be concerned with the increased cost of importing these aggregates and the difficulty in obtaining them.

One area in which the engineer-administrator must place increased emphasis is the reduction of frictional needs of traffic. The fuel shortage, which has caused a reduction in speed limits and traffic volumes, has been a blessing in disguise. We have already seen evidence of sharp reductions in fatalities over the Christmas and New Year weekends. It is impossible to determine how much of this reduction was due to the lesser amount of traffic on the highways and how much was due to the reduced speeds. Regardless, it appears that a major portion of the reduction must be attributed to the reduced speeds because neighboring states that had presumably the same reduction in total traffic volumes but have not reduced their traffic speeds have not enjoyed the same reduction in fatal accidents. When the fuel crisis is over, serious consideration must be given to maintaining these reduced speeds for safety reasons.

Other efforts must be made to further reduce the frictional needs of traffic. These are primarily related to geometric design and an improved knowledge of traffic operations, driver behavior, and vehicle characteristics.

A list of remedies for skidding risk would not be complete without a discussion of several possibilities that either reduce the frictional needs of traffic or decrease the frequency of their occurrence. Although AASHTO recommendations are largely followed in new highway designs, they should be applied also to existing highways to improve sight distances, reduce grades (particularly those near and at intersections), and provide superelevation where there is none. A systematic program to gradually improve road geometry, which, in contrast to skid-proofing, produces lasting results and very likely also reduces accidents that are not due to skidding is urgently needed. High-accident sites need to be identified and maintenance improved. The large number of skid marks in built-over areas clearly suggests that providing a single access to several business establishments along a highway might have advantages and certainly would reduce the frequency of brake applications, which cause skidding and increase collision risk. Drivers could do much to create a greater margin between friction availability and demand by reducing speed when pavements are wet and by starting early and executing smoothly vehicle maneuvers. More sophisticated driver training methods may provide a partial solution to this problem, but the development and use of all aids that feed needed information to the driver, protect him or her from distractions (particularly at locations and in situations that call for rapid decision-making), assist him or her to make the correct decisions, and help control the vehicle accordingly should be vigorously promoted. All such improvements reduce the danger of skidding and its consequences.

After we have done all we can on the pavement to improve its frictional characteristics and to reduce the frictional needs, we must go a step further and provide a clear roadside policy, and, I want to make a final plea that all concerned recognize the need for improvements in vehicle and tire design, vehicle inspection programs, and driver education programs.

REFERENCES

1. Carlson, R. F. A Review of Case Law Relating to Liability for Skidding Accidents. Paper in this Record.
2. Kummer, H. W., and Meyer, W. E. Tentative Skid Resistance Requirements for Main Rural Highways. NCHRP Rept. 37, 1967.
3. Farber, E., Janoff, M., Cristinzio, S., Blubaugh, J., Reisener, U., and Dunning, W. Determining Pavement Skid Resistance Requirements at Intersections and Braking Sites. NCHRP Rept. 154, 1974.
4. Meyer, W. E., Hegmon, R. R., and Gillespie, T. D. Locked-Wheel Pavement Skid Tester Correlation and Calibration Techniques. NCHRP Rept. 151, 1974.

BACKGROUND AND DEVELOPMENT OF THE FEDERAL HIGHWAY ADMINISTRATION'S SKID-ACCIDENT REDUCTION PROGRAM

D. W. Loutzenheiser, Federal Highway Administration

Review is made of Federal Highway Administration actions related to pavement skid-control qualities of federal-aid highway projects since the Highway Safety Act of 1966. Emphasis is on measuring skid resistance; obtaining and correlating accident data, material properties, and practices in design, construction, and maintenance for better skid resistance; and determining priorities for current projects. A coordinated research program was expanded in 1970 to include study on all phases of skidding accidents and the engineering factors and actions for their reduction, including 3 field test centers to calibrate skid measurement equipment. It is FHWA policy that pavement surfaces on all projects be designed, constructed, and maintained with the best practical skid resistance properties and that highway sections that are inadequate be identified and corrected. Since 1968, project work to improve pavement skid resistance has been eligible for federal aid. Concern of the state highway departments for the legal implications of use of minimum skid numbers is recognized. The present program uses guidelines set by each state for their specific conditions to establish priorities for correction.

•THE TOPIC of skid resistance has been around for a long time. As far back as the 1930s Ralph A. Moyer, formerly of the Highway Research Board, and others were expressing concern about safety problems and were presenting research reports on conditions and early measurement details. The 1958 International Skid Conference in Virginia was the first major effort to bring together the many engineering and research aspects of the subject. House Report 1700 (1966 accompanying H. R. 13290) identified skidding on wet pavements as 1 of the principal contributing factors in many accidents. The resultant Highway Safety Act of 1966 renewed emphasis at the federal level to improve pavement skid resistance. This continuing emphasis has resulted in what has been termed the federal skid-accident reduction program.

Do not be misled by the word "program," which has a wide variety of meanings. A Federal Highway Administration skid-accident reduction program exists not as a separately funded, specific set of projects but rather as actions toward a general goal. Better skid resistance is 1 of several safety features being emphasized in the federal program for construction and reconstruction of highways in the federal-aid system. It is also a significant part of the general highway safety program that applies to the many thousands of miles of streets and highways not in the federal-aid systems.

The stated policy of the FHWA is that pavement surfaces should be constructed and maintained for the best possible skid resistance and that inadequate pavements be identified and corrected. This calls for attention to materials, design, construction, operations, maintenance, research, development, and administrative controls on vehicles and drivers. On April 21, 1967, the FHWA issued a Circular Memorandum (CM),

Publication of this paper sponsored by Legal Resources Group Council and Group 2 Council.

Pavement Skid Resistance, to field personnel emphasizing the importance of providing good skid resistance on federal-aid roadways and stressing the need to include provisions for high skid resistance during the design and construction of federal-aid projects. In addition, the memorandum transmitted 2 documents. One dealt with the present state of the art on the causes and nature of skidding and skid resistance, and the other gave information on applying skid-resistant surfaces to new and existing pavements. At that time, minimum standards for pavement surface skid resistance had not been firmly established but obviously were needed.

In June 1967 the FHWA's National Highway Safety Bureau issued 13 highway safety program standards. These are uniform performance standards that each state must implement as part of its comprehensive highway safety program. It should be noted that these standards are not specific design details such as those developed by AASHTO on geometric items but rather broad goals and objectives to be attained through the program. The Standard on Highway Design, Construction and Maintenance included 2 important provisions on skid resistance that apply to all streets and highways.

(1) Every State, in cooperation with county and local governments, shall have a program of highway design, construction and maintenance to improve highway safety.

(2) The program shall provide as a minimum that: ...

  D. There are standards for pavement design and construction with specific provisions for high skid resistance qualities.

  E. There is a program for resurfacing or other surface treatment with emphasis on correction of locations or sections of streets and highways with low skid resistance and high or potentially high accident rates susceptible to reduction by providing improved surfaces.

In April 1968 the FHWA issued Instructional Memorandum (IM) 21-3-68, Construction of Pavement Surfacing to Provide Safer Coefficient of Skid Resistance, which pointed out the benefits of good skid resistance and encouraged improvement projects. More importantly, it informed the states that in the interests of safety, there would be federal-aid eligibility for work to resurface pavements with skid numbers (SNs) of less than 35. Up to this time, it had been a policy to view any of the several forms of thin overlays as a maintenance step to restore pavement and, therefore, there was no eligibility for federal funds. With this memorandum, surface courses provided primarily to improve skid resistance were accepted as needed safety improvements that could be funded. The SN criterion was based on data included in NCHRP Report 37 (1). This report is a good summary of the state of the art on pavement skid resistance. It presents tentative minimum SNs for main highways.

On May 13, 1968, the FHWA issued a followup CM, Plant Mix Seal Coats, to better describe plant mix seal coats and encourage states to use them to provide high skid resistance on existing pavements.

Since early 1969 there has been an FHWA Highway Safety Improvement Program as outlined in Policy and Procedure Memorandum (PPM) 21-16, which was last revised on May 3, 1972. This is a program of projects to detect and correct hazardous or potentially hazardous locations, elements, or sections of the federal-aid highway system. Accident data are used to identify hazardous spots. Continuing systematic corrections and evaluations are required for skid-prone locations.

During the summers of 1967 and 1968 the FHWA conducted a program of pavement skid testing with the cooperation of 17 states using a BPR skid-test trailer. The 2 main objectives of the program were to illustrate the ease, efficiency, safety, and reliability of a standard skid trailer to conduct rapidly a large number of skid tests in various environments and to obtain widespread technical data on existing pavement skid resistance. Two reports, 1 to AASHO and 1 to the HRB, were given in 1968 on the data collected in 14 of the states.

By 1970 it became evident that there was a need to develop a comprehensive approach to the skid-accident problem. A number of state studies indicated a dramatic reduction in skidding accidents after overlaying or grooving of existing pavements with low SNs. Also, increasing evidence showed that many highway sections had less than desirable skid characteristics and that high speeds and high traffic volumes were polishing pavements at a rapid rate. Formulation of a major research program was started. A series of regional workshops on skid-resistant surface courses was begun. A number of meetings were held among the affected safety, operating, and research offices within FHWA to develop a broad approach to aid all states. A special ad hoc committee was formed to obtain and analyze all known information on skid resistance and to develop a comprehensive program to reduce skidding accidents.

Through the efforts of this special committee, a proposal for a skid-accident reduction program was developed in July 1971. This was commonly referred to as the "white paper" on skid resistance. Although the white paper was not used directly as FHWA policy, it was the basic resource used to develop the current control memorandum on skid-accident reduction. With the data then available the committee felt that enough information was available for the states to develop an initial program that included 3 main action features:

1. Countrywide detection of pavements with low skid resistance through inventories and accident analyses;
2. Systematic correction of existing pavements that have become low on the skid resistance scale; and
3. Development of better design criteria to both provide and retain better skid resistance in new highway construction.

To support this program, the committee also identified areas in which further research was needed. For example, different correlation studies indicated that skid trailers owned by different states were not producing similar SNs. Wide studies on related research needs led to a major FHWA-correlated research activity with a goal to reduce the frequency and severity of skidding accidents. The project was divided into 8 sections:

1. Materials and techniques for durable skid-resistant surfaces;
2. Mechanics of pavement-vehicle interactions, skidding, and loss of control;
3. Measurement systems for pavement friction, roughness, and hydroplaning potential;
4. Friction requirements for highway sites with high skid-accident potential;
5. Skid-accident analyses and identification of highway sites with high skid-accident potential;
6. Driver awareness of highway sites with high skid-accident potential;
7. Cost effectiveness of alternate solutions for highway sites with high skid-accident potential; and
8. Procedures, equipment, and test centers for standardizing skid measurements.

Progress has been made on all of the tasks and 3 centers have been established for calibrating and certifying measurement equipment.

In March 1971 the National Highway Safety Bureau of FHWA issued Volume 12 of the Highway Safety Program Manual, which supplements Standard 12, Highway Design, Construction and Maintenance, and suggests guidelines for implementing the various features of the standard. The manual calls for a program under which any pavement that cannot meet the "recommended minimum interim skid numbers" [values from NCHRP Report 37 (1)] should be considered for corrective action. It also states that specific consideration be given to skid resistance qualities in the materials, design, and construction of new pavements.

Meanwhile, the Subcommittee on Investigations and Oversight of the House Public Works Committee expressed disappointment at the slow progress being made in skid resistance and scheduled hearings in May 1971. A film used during these hearings showed highly disturbing skidding incidents on Interstate segments in the Washington, D.C., area. The subcommittee charged the FHWA to give the states leadership in the prevention of skidding accidents. They urged that we encourage better state response on testing, reporting, and improving pavement skid resistance. The subcommittee also indicated that future hearings on our progress would be held. The subcommittee scheduled a hearing for the spring of 1973, but it was postponed.

In 1972 the FHWA ad hoc committee was reassembled to give policy instruction to our field offices and the states. After much deliberation a draft memorandum was prepared and distributed to all affected units within the FHWA. The nature and extent of specific FHWA requirements for state actions were heavily debated. Some felt that we should call for correction of all highway sections with SNs below a named value. Others, who reflected the states' concerns about widely varying conditions, lack of widespread inventory data, and legal implications, believed that only something less definite would be workable. We resolved these divergent views in a few months and issued a revised draft that we hoped would be acceptable to all factions of the highway community. This draft contained a table of minimum SNs, below which corrective work would be required. These minimum SNs were based on those from NCHRP Report 37 (1). The memorandum also included a guide that contained a table of desirable SNs and speed gradient characteristics for new constructions.

To ensure acceptance of this controversial IM we furnished copies to the AASHO "Red Tape" committee for comments. During this time, all of the states received copies and replies were received from 20 of them, which indicates the great interest in this problem. Comments ranged from complete endorsement of the proposed IM to complete rejection with the suggestion that we avoid all program requirements and leave the matter entirely to the states. One consistent comment on the SN guidelines related to the possible legal liability of a state if highways did not meet the minimum SNs. Because some states had lost suits on unsafe highway conditions they were reluctant to officially collect data such as skid inventories and listings of hazardous locations that could be used in the courts against them. In response to the states, we once more revised the IM and eliminated all direct references to minimum SNs. We also asked that each state set up its own general guidelines based on specific conditions. These guides are expected to help establish program priorities for skid reduction projects.

The current FHWA program is described in the July 19, 1973, IM, Skid Accident Reduction Program. Its main contents are: (a) a policy statement; (b) the essence of Highway Safety Program Standard 12 restated as a call for state programs for evaluation of current pavement design, construction, and maintenance practices and for systematic identification and corrections; (c) a reference to PPM 21-16, May 1972, Highway Safety Improvement Program, to amplify accident data studies with pavement-friction data; (d) requests for more complete state skid measurement data and appraisals of present practices; (e) a request for engineering evaluation of skid, geometric, traffic, weather, and other factors for corrective action; (f) a statement about federal-aid fund eligibility; (g) a request for progress reports; and (h) guidelines on the technical details of evaluation, design, and construction for skid resistance.

We believe that the inventory work required by the Highway Safety Program Standard 12 can be used for the general skid reduction program. Initial measurements should be made on selected samples of surfaces representative of the various combinations of mix designs, aggregates, and construction procedures to appraise skid resistance. This information can then be used to estimate the condition of similar pavements and to determine probable critical locations. A location in need of a thorough engineering evaluation for corrective action may be identified by a high frequency of wet weather accidents, by a low SN, or by a combination of the 2.

The FHWA has determined that federal-aid funding can be made available to programs to provide new or reconstructed pavements with desirable skid resistance qualities. Work that is justified solely by skid resistance measurements is limited to corrective treatment of the pavement surface. This work may consist of grooving the surface or applying a thin overlay of bituminous material specifically designed to provide the desired skid resistance.

Our division engineer is expected to monitor the states' skid resistance improvement program on a continuing basis, review it for reasonableness and consistency, and see that it is implemented at the earliest possible date. The engineer is not to approve future project plans involving surface courses unless these surfaces will satisfy the objectives of the IM. In short, we insist that future projects be done so that better skid-resistant surfaces will be provided for the traveling public.

The FHWA skid-accident reduction program described in the July 19, 1973, IM 21-2-73 is an expansion of the ongoing highway safety improvement program (PPM 21-16) for spot improvement projects, which has been under way for about 4 years. It is also concerned with better skid quality design and construction of new highway projects. It is not a separately funded program, but rather an integral part of the overall federal-aid highway program. The program required some time to develop and represents a compromise between the high safety desires and practical limitations that exist today.

The nature of the program is a compromise and it recognizes several deterrents against more rapid or more widespread pavement skid betterment. These include:

1. Priority scrambles for use of available highway funds for the most cost-effective returns;
2. Incomplete development and acquisition of rapid, reliable skid measurement equipment, and a resultant lack of pavement condition data;
3. Inertia in agencies making changes in materials, design, and construction practices;
4. Limited supply of high-quality aggregates and incomplete technical testing knowledge of those available; and
5. Highway agency concern about liability to lawsuits for accidents caused by skidding.

Obviously, the legal implications of highway skid resistance are and will continue to be significant factors in highway safety programs. But, there are others too. The FHWA will continue to press for adequate and up-to-date programs that will reduce skidding hazards.

REFERENCE

1. Kummer, H. W., and Meyer, W. E. Tentative Skid Resistance Requirements for Main Rural Highways. NCHRP Rept. 37, 1967.

DEFENSE AND SETTLEMENT OF CLAIMS FOR SKIDDING ACCIDENTS

William B. Somerville, Smith, Somerville, and Case, Baltimore, Maryland

Statutes of limitation of 43 states and the District of Columbia may bar a claim against a highway design engineer. If the design engineer is not protected from claims of third parties by state statute, he or she may have no liability if the design and the structure were completed and accepted by the owner. In determining whether the design engineer may be liable to claims of third parties, that is, the traveling public, 5 questions are to be evaluated. Did the design engineer owe a duty to the public? Was the duty continuing in nature? Was there a breach of the duty? Did the breach of the duty constitute a "public nuisance"? And, did the breach of duty cause or materially contribute to the events giving rise to the claims? Should a claim be made against a design engineer as the result of a skidding accident, drawings, records, and files are the most important defense tools. He or she should always retain the owner's instructions, his or her calculations, and research into design criteria on each project because this material is the record that will determine whether he or she will be exposed to liability.

•BOTH Carlson (1) and Gartner (2) discussed sovereign immunity and the changes that have occurred over the past 10 to 15 years. There are, of course, some states such as Maryland where the sovereign is still immune. But, even if the state is not liable, the engineer could be. If an engineer is sued for damages as the result of a skidding accident, his or her attorney first will look to see if the suit is barred by a statute of limitations. Until the Wisconsin legislature passed a statute in 1961, the design professional was subject to suit for a period of time after the event occurred, even if the facility may have been designed and constructed 20, 30, 40, or more years before the incident. With the enactment in 1973 of a statute in Wyoming, 43 states and the District of Columbia have statutes of limitations for periods as short as 2 years and as long as 20 years. Eighteen states have a 10-year statute. Most of these statutes provide that legal action is barred unless the injury occurs within "X" years beginning on the date the facility was substantially complete—when it was first available for its intended use. A list of existing statutes of limitations is given in Table 1.

If action is not barred by time, the engineer's attorney will turn to case law for the state or states where the design contract was signed, the design prepared, and the construction accomplished. An attorney's steadfast wish is to find a "case on point" which will lead to a prompt decision as to whether the case should be defended or settled. There is a dearth of case law specifically discussing the liability of an engineer for defective highway design. Thus, an analysis of prospective liability that might arise out of automobile skidding accidents must proceed on a traditional "duty, breach, causation" framework in terms of related existing case law.

Initially, there is the issue of whether an engineering firm that designs a highway owes a duty to the public. The trend of authority appears to be toward imposing the same duty on the engineer as is imposed on the manufacturer of a chattel, pursuant to

---

Publication of this paper sponsored by Legal Resources Group Council and Group 2 Council.

Table 1. Existing statutes of limitations.

State	Year of Passage	Statutory Period (years)
Alabama	1969	4
Alaska	1967	6
Arkansas	1967	5
California	1967	4
Colorado	1963	10
Connecticut	1969	7
Delaware	1970	6
District of Columbia	1972	10
Florida	1967	12
Georgia	1968	8
Hawaii	1969	6
Idaho	1965	6
Illinois[a]	1969	6
Indiana	1967	10
Kansas	1963	10
Kentucky	1966	5
Louisiana	1964	10
Maryland	1970	20
Massachusetts	1968	2
Michigan	1967	6
Minnesota	1965	10
Mississippi	1966	10
Montana	1971	10
Nebraska	1972	10
Nevada	1965	6
New Hampshire	1965	6
New Jersey	1967	10
New Mexico	1967	10
North Carolina	1963	6
North Dakota	1967	10
Ohio	1963	10
Oklahoma	1967	5
Oregon	1967	10
Pennsylvania	1966	12
South Carolina	1962	10
South Dakota	1966	10
Tennessee	1965	4
Texas	1969	10
Utah	1967	7
Virginia	1964	5
Washington	1967	6
Wisconsin	1961	6
Wyoming	1973	10

[a]Earlier statute of 4 years declared unconstitutional by Illinois Supreme Court in Skinner v. Anderson, 231 N.E.2d 588.

§ 398 of the Restatement of Torts, Second. In a recent article, this author commented on the area of defective highway design as encompassing a "malpractice action against the engineers themselves."[1] However, there is no case law that, on this precise question, imposes on the engineer a duty to the traveling public, although some cases have skirted the issue. In Rigsby v. Brighton Engineering Company, 464 S. W. 2d 279 (Ky. 1971), an action was brought on behalf of the estates of occupants of an automobile who were killed in a crash against a bridge pier. This action was brought against the engineering consultant who designed the highway and was based on failure to recommend installation of guardrails around the bridge pier. Summary judgment for the defendant was affirmed on appeal on the ground that the state agency had adopted the criteria that were binding on the defendant consultant. Although the Court of Appeals of Kentucky cited no precise case law, it is clear from a reading of the opinion that the court felt that without such state recommendation the engineering consultant would have been held liable for defective design.

Somewhat to the contrary is the decision in Black v. Peter Kiewit Sons' Co., 497 P.2d 1056 (Idaho 1972). In that case a negligence action was asserted against a highway contractor on the grounds that an oil slick on the highway section constructed by the contractor caused the plaintiffs' automobile to skid and go out of control. In affirming summary judgment for the defendant, the Supreme Court of Idaho also noted that the highway was constructed in accordance with state specifications. However, the court recited the older traditional view that [497 P.2d at 1058 (65 C.J.S. Negligence § 95 at 1060-1062)]

> Where the work of an independent contractor is completed and is turned over to, and accepted by the owner, the contractor is not liable to third persons for damages or injuries subsequently suffered by reason of the condition of the work; the responsibility if any, for maintaining . . . the property in its defective condition shifting to the owner.

This general notion of acceptance of the highway as releasing all prior independent contractors from responsibility was also given in Williams v. Sullivan, Long and Hagerty, Inc., 209 So.2d 618 (Miss. 1968). There, a motor scooter struck a hole in a street that had been paved by the subcontractor defendant. The court held that there was no continuing duty on the part of the defendant after the road was accepted by the county.

However, the older "acceptance" notion has not been an absolute shield to independent contractors in highway construction. The case of Henry v. Haloatt Construction

[1]Kenneth Barranger, Liability for Negligent Highway Design: The Louisiana Perspective, 20 La.B.J., 277 (1973). See also Steven J. Erlsten, Defectively Designed Highways, 16 Clev.-Mar.L.Rev. 264 (1967).

Co., 488 P.2d 1286 (Okla. 1971), contains dicta to the effect that, despite the general "acceptance" rule precluding an independent contractor's liability, in those cases in which a contractor creates a dangerous condition that he or she knows is immediately dangerous, his or her duty to the general public may continue. The dicta on continuing duty in this Oklahoma case lead us to an analysis of the Maryland law that presents a similar theory.

The case of Cumberland v. Turney, 177 Md. 297 (1939), appears to be the inaugural opinion by the Maryland Court of Appeals on defective highways and automobile accidents. In this case, an automobile skidded off the road and crashed into a pole when the driver attempted to follow a curve in the road. An injured infant passenger in the automobile brought suit against the city of Cumberland, alleging defective design in the highway and particularly contending that the smoothness of the surface of the highway constituted negligence on the part of the municipality. The Court of Appeals reserved a lower court judgment in favor of the plaintiffs, finding that there had been no negligence on the part of the municipality. The court found no evidence of negligence, "in view of the facts that the surfacing material was selected by experienced engineers."

The court did not speak to the question of any negligence of the engineers who had selected the surfacing material. But, in the case of East Coast Freight Lines, Inc. v. Consolidated Gas Company, 187 Md. 385 (1946), the Court of Appeals approached the subject of design of a street and the duties of an independent contractor in relation to that street.

The Consolidated Gas case involved an action against a trucking company and arose out of a serious automobile accident that was caused by a vehicle's left front wheel hitting the curbing around a grass plot, striking a lamppost, and colliding head-on with another vehicle. The defendant trucking company asserted a third party complaint against the gas company, alleging that the lamppost was an obstruction of the highway, and as such was a public nuisance, and that, therefore, the defendant gas company owed a duty to the public to warn of the lamppost. Because the lamppost had been purchased by the city of Baltimore, and its location had been determined by the city, the issue essentially was whether the gas company owed any duty to the traveling public. The Maryland Court of Appeals sustained the demurrer of the gas company to the third party complaint, but in important dicta recognized (187 Md. at 397) that

> A contractor, even after he had completed his work, may be held liable in damages if such work is inherently dangerous and constitutes a public nuisance.

An electric light pole was not seen to constitute such a nuisance. A few years later, the case of State v. Prince George's County, 207 Md. 91 (1954), held that the municipality would not be held liable for every irregularity in the grading of the street. Thus, at this point in the common law of Maryland, it did not seem that the surfacing of a street could constitute a nuisance.

However, this line of relevant Maryland case law appears to end with the litigation of Jennings v. United States, 178 F. Supp. 516 (D. Md. 1959), remanded 291 F.2d 880 (4th Cir. 1961); 207 F. Supp. 143 (D. Md. 1962); aff'd 318 F.2d 718 (4th Cir. 1963). The Jennings cases involved an automobile that skidded on a patch of ice on Suitland Parkway, a Maryland road maintained by the National Park Service. Serious injuries and death ensued from the resulting collision, and suit was filed against the United States under the Federal Tort Claims Act. In the initial suit, Judge Watkins rendered judgment against the United States, finding that the government should have discovered and corrected the unsafe condition. The Fourth Circuit Court of Appeals reversed that decision, holding that proof of mere skidding did not support the judgment. The Court particularly stated (291 F.2d at 887):

> In Maryland it has been held that there is no liability for injuries caused by a design defect in a highway, but if a defect, whether of a design or not, creates a condition which would itself constitute a nuisance, and reasonable care to abate it, is not exercised and the condition is the effective cause of the injury, no reason presently appears why the agency charged with maintenance of the highway should not be responsible as for any other nuisance it unreasonably permitted to exist.

The case was remanded for a finding on that issue. On remand, Judge Watkins held that the United States was liable, because inadequacy of the drainage system had caused the ice to accumulate. The court found (207 F. Supp. at 144) that

> Suitland Parkway, at the time in question, was defective, both in design and construction, and is so constructed and maintained that it constituted a nuisance.

The Fourth Circuit affirmed this finding. Thus, Maryland case law holds that an independent contractor may be liable for creating a public nuisance. Additional case law indicates that a defectively designed highway can constitute such a public nuisance.

Given the modern trend of authority, and the Maryland common law in the area of nuisance, it appears that a plaintiff in an action against an engineer for a defectively designed highway that caused an automobile to skid could establish that there was a duty owed to the public. One could first proceed under the view of a direct duty owed, and could secondly proceed on a nuisance theory, which has been recognized by the Maryland Court of Appeals and the United States District Court of the District of Maryland. On this question of duty, however, there is a subsidiary question on whether that duty is a continuing duty. Although state acceptance of a defectively designed highway will not terminate an engineer's duty as to proper design, it imposes on the state the duty of maintenance. Indeed, this duty was commented on in the Jennings decision. In the decision of Baldwin v. State, 491 P. 2d 1121 (Cal. 1972), the state was held not to be immune from suit for defective highway design in which it failed to construct a left-turn lane. The California court found that, when a governmental entity has notice of the dangerous condition of a highway, it must act to alleviate such a condition. Nevertheless, it should be noted that the case of Rush v. Pierson Contracting Co., 310 F. Supp. 1389 (E.D Mich. S.D. 1970), reached a contrary result. In that case, the motorist was deemed to have a cause of action for damages against the dependent contractor 5 years after the work had been accepted by the state.

The distinction between an initial duty owed and a continuing duty becomes significant when we turn to law that might determine whether there has been any breach of duty in a skidding accident. There does not seem to be any case law that discusses the particular breach of a duty by an engineer concerning road surfacing and skidding accidents. Thus, an analysis must be of those cases that initially have held that a governmental entity was not immune from suit, and subsequently met the question of whether there was liability for defective highway design, which resulted in a slick surface causing the skidding of an automobile. In Carthay v. County of Ulster, 168 N.Y.S.2d 714 (1957), an action was brought against the county for injuries sustained in an automobile accident when the automobile skidded on a wet road at a point where there was a sharp right curve in the road. The trial court rendered judgment against the county, and, on appeal, a New York appellate court affirmed by holding that the evidence sustained a finding that the road was negligently maintained. A contrary result was in the New York case of Lidell v. State, 236 N.Y.S. 2d 1005 (1963), which also involved a similar action against the state arising from a skidding accident. Again, the issue was that of maintenance by the state, and the court found that the state of New York had done "all that it could" to protect the public. A similar situation with a sharp curve and skidding accident was in Clary v. Polk County, 372 P.2d 524 (Ore. 1962). An alleged defect in the road was the existence of a slick oil surface. Liability was imposed on the county for such a defect pursuant to an applicable Oregon statute. This case did not specifically employ the concept of maintenance, but the court cited related cases, all of which dealt with the question of negligent maintenance by state authorities.[2]

---

[2]See also Foley v. State, 203 N.Y.S.2d 196, 214 N.Y.S.2d 665 (1960) (variation in banking on the curve, failure to maintain adequate warning signs); LeBoeuf v. State, 11 N.Y.S.2d 640 (1938) (slippery surface due to maintenance of macadam surface); Lusk v. State Highway Department, 186 S.E.786 (S.C. 1936) (loose sand on surface, inadequate warning signs); Rubell v. Santa Clara County, 80 P.2d 1023 (Cal. 1938) (loose gravel on road, failure to warn); Sporborg v. State, 234 N.Y.S. 476 (1929) (skidding of automobile, negligence of state in not having warning devices on the premises).

An analysis of this case law indicates that in all those situations where liability was imposed on the state there was a breach of the continuing duty of maintenance. The governmental body was under some duty to maintain the road as constructed and warn of dangerous areas. None of these skidding cases discusses any initial liability for the road surface used in the original construction. Even the decision in Clary v. Polk County, supra, does not discuss initial duty as such. In terms of existing case law, it seems that duty must be of a continuing nature for a breach to be found. Although the plaintiff in such a skidding action can proceed on the "nuisance" theory, it should be noted that the Jennings litigation also was based on the defective maintenance of Suitland Parkway. On this distinction between initial duty and continuing duty, Judge Northrop's opinion in Mondshour v. General Motors Corporation, 298 F. Supp. 111 (D. Md. 1969), should be noted. A child was injured when trapped under the right rear wheel of a bus leaving a curb. Because the bus did not have a right rearview mirror, the plaintiff brought action against the manufacturer of the bus, alleging that the bus was defectively designed. Judge Northrop noted that, even if General Motors had been negligent in its original design, the negligence of the Baltimore Transit Company in failing to maintain its equipment to meet changing conditions "would be a superseding intervening cause" (298 F. Supp. at 114). The skidding cases that have resulted in state liability have done so in terms of a breach of a duty to maintain the highways in light of changing conditions. This duty would seem to be incumbent on the state and not the designing engineer.

It might be argued that the engineer shares part of this continuing duty of maintenance with respect to keeping the state advised on possibilities for redesign. Thus, such a continuing duty would bring the engineer within many of the skidding cases discussed above. Only 1 relevant case has been found for this argument. The case of Natina v. Westchester County Park Commission, 158 N.Y.S.2d 414 (1966), involved a "cross over", head-on automobile accident. There are dicta in that opinion to the effect that recommendations made for highway redesign are not matters of maintenance.

After the initial hurdles of establishing a duty, and then a continuing duty, and finding a breach thereof, the plaintiff in a skidding case will obviously have to establish proximate cause. On this question, there are 2 recent New York cases that indicate the problems a plaintiff must face in this area. In Cruz v. State, 327 N.Y.S.2d 889 (1972), the automobile went off the paved portion of the highway and crashed into a blockhouse apparently 4 ft off the paved portion of the road. One of the alleged defects was improper grading of the road. The trial court dismissed the case, holding that any negligence of the state was not the proximate cause of the accident. On appeal, the New York appellate court affirmed, holding that (327 N.Y.S. 2d at 891)

> [W] here there are several possible causes of an accident, for one or more of which the State is not responsible, the claimant can not recover without proving that the injury was sustained wholy or in part by a cause for which the State was responsible.

The case of Stuart-Bullock v. State, 326 N.Y.S.2d 909 (1971), involved similar facts; it was alleged that parts of the road surface could have caused the decedent's car to veer off the road. A judgment in favor of the claimant was reversed, and the appellate court noted that "the trial court (had) engaged in pure speculation" (326 N.Y.S.2d at 911). The court held that the claimant had simply failed to establish proximate cause.

As was indicated initially, there is not a wealth of case law on the liability of an engineer for defective highway design with respect to skidding accidents. Yet, analysis of the reported cases on state liability and duties owed by independent contractors indicates several "roadblocks" in the path of a plaintiff who sues on the basis of such a defect. Although an initial duty on the part of an engineer can be established, unless the highway is designed so defectively as to initially constitute a public nuisance, the plaintiff must establish that there was a continuing duty owed to establish any breach by the engineer. This is certainly the tenor of the cases that have imposed liability on governmental authorities. Further, proximate cause in skidding accidents might prove more difficult to establish than in cases that proceed on the basis of absence of barriers. Certainly, the 2 recent New York decisions indicate the difficult burden on the plaintiff.

In short, in any suit against an engineer alleging defective highway design that has caused the skidding of an automobile, the plaintiff may find the road to recovery difficult to travel.

In considering this road to recover damages against an engineer, the attorney must decide at an early time whether the claim should be defended or settled out of court. If a claim is timely, it may be that an early appraisal of the facts will dictate the good sense of a prompt settlement. Frequently "good sense" has a hard time emerging from the barrier of "pride of design," which sometimes leads to the conclusion that "not only is my design without fault, but our firm is insured and we expect our insurance company to protect us."

No specification has been written for the successful defense of every claim resulting from a skidding accident. There are a number of steps to be taken and things to be done by the engineer and his attorney. First and most importantly, keep the files and drawings on each job until you are sure that you are protected by statutes of limitations in every jurisdiction that might apply. Second, refuse to depart from a design that you can defend unless you have a written order by the owner. Third, do not try to be the contractor. (I should add a fourth, keep your insurance premiums paid.)

And, trust the attorney defending the case and give him or her your time and cooperation, even if the demands appear to be burdensome to the point of exasperation.

REFERENCES

1. Carlson, R. F. A Review of Case Law Relating to Liability for Skidding Accidents. Paper in this Record.
2. Gartner, W., Jr. Engineering and Administrative Considerations in Constructing, Maintaining, and Testing Skid-Resistant Pavements. Paper in this Record.

LEGAL IMPLICATIONS OF HIGHWAY SKID RESISTANCE

At the end of the third session of the symposium those who presented papers answered questions from the audience. This is an edited transcription of the session. The moderator was W. A. Goodwin, University of Tennessee at Knoxville. The panelists were Robert F. Carlson, California Department of Transportation; William Gartner, Jr., Florida Department of Transportation; D. W. Loutzenheiser, Federal Highway Administration; and William B. Somerville, Smith, Somerville and Case, Baltimore, Maryland.

Question

What is the necessary extent of the engineer's compliance with the state of the art?

Somerville

A highway designed today could be perfectly adequate for present and anticipated future needs and could be an example of good engineering design. But, in 15 years, that original design may not meet the state of the art. If there were an accident and there were no statute of limitations, the engineer should not be liable. There is, however, a question of the continuing duty, if there be such, of the design engineer to follow the project and make recommendations for changes in the design of the facility.

Question

It is my opinion that we do not know what the minimum available coefficient of friction should be. Do you agree? What is needed for wet weather driving?

Goodwin

Is the question that you do not feel that we know what the minimum coefficient ought to be?

Comment

Not only that but that we do not know what it is.

Loutzenheiser

I presume you are talking about the use of a single factor on a countrywide basis. Total conditions are known to be sufficiently different that we cannot expect to describe them with a single skid number and say that everybody every place ought to fix highways that are below it. There is just too much variability in the physical part of the highway, the weather, tires, drivers, and many other things. I doubt that we can ever arrive at a single factor. Within the practical realm of what we have now it certainly is possible for each state or agency to determine, on a relative basis, at least, the bad spots and fix them. This calls for flexible criteria depending on the amount of money available and what we are able to do. A substantial effort should be based on what we can do today, but it would not be based on a uniform standardized criterion.

Publication of this paper sponsored by Legal Resources Group Council and Group 2 Council.

Comment

I agree with your comment that many states do not have enough money to bring all of their highways up to a minimum skid number at once. Texas would probably take 2 or 3 yearly programs for this project alone and would have to shut down everything else.

Question

What is the statute of limitations for Texas?

Somerville

Texas has had a statute of limitations since 1969 and it is 1 of the 18 states that has a 10-year statutory period.

Comment

You ought to let them go ahead and sue the states at fault. We do not have any money anyhow.

Gartner

We have 1 inspector in Florida who makes about $500 or $600 per month who just got sued for $100,000. I do not think he has the money, and he does not feel happy about being sued.

Carlson

In California we have a specific statute for the occasion when an employee is sued for damages that arise out of the scope and course of employment. By it, the state is required to provide that employee with a defense and to pay any settlement or judgment. I do not believe that an engineer's personal fortune should be put on the line if he or she is sued individually when the plaintiffs should really be suing the state.

Question

Most states report that the state highway department could be held liable in the absence of warning signs. But, the presence of a warning sign announces that the state is aware of the dangerous conditions, does it not?

Carlson

Warning signs are a means to prevent the imposition of liability on a government agency. Also, by putting up a warning sign you are admitting, at least in California, that the highway may be dangerous when wet. I believe that warning signs are not the remedy because we are still going to have accidents. I firmly believe that we should be preventing these accidents by using other remedial measures such as grooving to increase the coefficient of friction. We have warning signs to limit government liability, but we also want to prevent accidents.

Gartner

Department officials are caught in the conflict of putting up warning signs and thereby exposing themselves to liability. But, they also have a responsibility to the public to identify any hazardous condition. In that respect I think it is the duty of highway engineers, if they are aware of hazardous situations, to do 2 things immediately. First, make everyone else aware, and, second, take corrective action. I think that if you do both of these things in a reasonably prompt manner you will do more to protect yourself and motorists than you would by doing anything else.

Question

We have a county engineer who refuses to develop accident spot maps for his county because he feels that this will increase his liability. Do you care to comment on that?

Carlson

There is nothing wrong in the mere developing of accident frequency maps for specific locations because accidents are caused by many reasons other than the highway itself. Many plaintiffs' attorneys in California ask for printouts of our summaries of accident locations and we give them to them. But I do think that to develop a safety program you have to have an accident profile on all parts of your highway system. This is 1 way to prevent accidents, and this information can help you as well as hurt you.

Question

I assume that the law makes no real distinction between the responsibilities of the state and those of lesser governments. What litigation has involved counties and municipalities?

Carlson

The California liability law applies to all levels of government agencies from the state down. The same law applies to every government agency with respect to liability for dangerous conditions on state highways, city streets, and county roads.

Question

How about the number of suits? Is it ordinarily the state that is being attacked or is it more frequently the counties?

Carlson

I have not personally checked the statistics of claims and volume of litigation against cities and counties but, according to attorneys doing defense work in this area, the number of claims and lawsuits is substantial. I don't know how this volume relates to the amount of litigation against the state.

Question

What liabilities are associated with full-scale field experiments designed to improve the state of the art?

Goodwin

The question is, What responsibility does the engineer have during a full-scale test on a public highway?

Carlson

We have conducted dynamic tests on a highway under the supervision of the expert witness who was going to testify on our behalf in a skidding accident case. If it is a high-speed freeway it is very difficult to shut down part of that highway to conduct these dynamic tests. The engineer and the state could be liable in conducting this type of testing. After an accident, we have conducted these tests at an abandoned airport site with the same type of vehicle, with the same type of tires, and, presumably, with the same tire inflation on pavement that has the same characteristics as the highway in the accident.

Gartner

I thought that the question related to pavement tests in which different types of pavement surfaces are put down and some surfaces turn out to be more slippery than the original pavement. In my opinion, when you conduct such an experiment you involve yourself in a certain amount of risk. The only thing you can do is to make motorists aware that you are conducting experiments and get the data as rapidly as you can.

Question

What is the liability if maintenance is required and the request for appropriation is denied by the legislature?

Carlson

We have in California the defense of what we call "reasonableness," the reasonableness of the action of the state, the state highway department, and its employees. In a case in which money was the answer to this problem and we could not get it from the legislature, we would use reasonableness as a defense. We have a problem with icy bridges in California. One of the best ways to deice a bridge besides salt is to put heaters in the bridge deck to automatically come on when the bridge reaches a certain temperature. We do not have enough money to take care of all the bridges that could use such a feature. The answer depends on the law in each state.

Somerville

It goes back to the New York case in which the court found that the state had done all that it possibly could, and that becomes a finding of fact. You know that on toll facilities there is a maintenance budget. If you do not have anything in it to do a particular piece of work I suppose you put up a sign that says "out of funds" or something like that. It gets down to the fact question, Has the state done all it could possibly do in good sense?

Question

We have had instances in our state in which expert witnesses testified against us who, we felt, were not competent in their field. Yet, it is the judge who determines whether a witness should be allowed to testify. I wonder if you can give us any direction about how the defense should go about getting an expert witness disqualified?

Somerville

One of these days we are going to be faced with the new federal rules of evidence. When they come into effect almost anything goes. I can be an expert or you can; all you have to do is say that you are an expert. Then it comes down to testing the expert, and the real tools are the publications of the Transportation Research Board. If a so-called expert witness' criteria are not found within the great volume of work that has been printed by the TRB, then I think that is a good attack. If a person is testifying today about what was good about a design long ago, then you can go back as far as you wish with material that has been printed by the TRB. Another method to use to attack a witness you think might be a fraud is to look around to see where this adverse witness has testified before. Get transcripts of testimony or depositions because the truth will catch up to a witness who testifies a lot.

Question

Do some of the states have an unfortunate tendency to treat letters from citizens complaining about a particular highway problem as just somebody else making a noise?

Carlson

That is a very good point. Notice can come from sources other than public employees connected with the highway. It can even come through editorials in the paper about a "blood alley" or some dangerous section of highway; it can come through letters to the editor; and it can come directly from letters from concerned citizens to the state highway engineer. These letters should not be treated lightly. They can be the foundation of the next lawsuit against the state government for not taking care of the matter. Because the government tends to keep all these letters on file, if nothing has been done about the situation mentioned in the letter, you are in a very bad position. Our instructions are to answer the letter if necessary and, if not, to at least put something in the file to indicate that an engineering decision was made regarding the complaint.

Question

Is the Florida Department of Transportation involved directly in driver education?

Gartner

No. The Department of Transportation is not, but the Department of Highway Motor Vehicles and Safety is. They are the branch of our highway patrol that issues driver's licenses. I have discussed driver education with the director of the Department of Motor Vehicles and Highway Safety and he has agreed that we need to put more emphasis on wet weather driving. This training generally is not given in non-snow-belt areas.

Question

Is the Federal Highway Administration going to get involved in driver education?

Loutzenheiser

Under §402 and 403 of the 1966 Highway Safety Act funds go to the governor and, through him, to state organizations. Driver education is not a part of direct federal-aid highway funds. But there is a federal program of funding that can be used for driver education.

Goodwin

Standard 304 of the National Highway Traffic Safety Administration requires driver education.

Question

The previous sessions have been devoted to attaining and maintaining minimum skid numbers. If you attain the recognized minimum skid number and there is a skidding accident, does this constitute a degree of defense or a valid defense in a lawsuit?

Carlson

Skidding accidents can happen even though the pavement has met the established minimum coefficient of friction. But, it is a good defense because the state has done all that is reasonable under the particular circumstances. The duty of care to that driver will be determined by the court or the jury. Just the fact that you have reached that magic number does not mean that you will not be sued or that there may not be liability.

Somerville

The mere fact that you have not broken the law is not necessarily a defense in a suit for damages. As with anything else, other pertinent facts could undermine your case.

Question

It is my understanding that according to the Supreme Court only a jury can determine what is reasonable. Would you care to comment on that?

Carlson

Reasonableness is a word used by lawyers to cover up and excuse their thinking when they do not know which way to go. To engineers everything is black or white; to lawyers everything is gray and in that gray area lawyers always use the term reasonableness. But you are right, the jury determines the reasonableness of the case. When they are determining reasonableness they are second-guessing the engineer.

Question

It seems that FHWA or possibly ASTM established a set of minimum skid numbers with the idea that federal funds had to be considered. It might be important to view that these numbers were not rigid enough, that they are still not safe. Could FHWA or ASTM be in some way liable for the establishment of specifications that were not adequate?

Gartner

And, Loutzenheiser mentioned that we can get federal aid if the skid number is 35 or lower. If it is 36 or 40 we cannot get federal aid for that particular job. That adds on to the question.

Carlson

That is a very difficult question to answer—what the liability of the federal government for establishing inadequate coefficient of friction would be. I would say, at least based on my knowledge of California law, that there would be no liability for the setting of a standard. It is like a promulgation of a law and would have a similar effect. When it comes down to money there is a real problem. A jury may second-guess you and say, Why did you spend $100,000 on that landscaping job when you should have spent the money grooving the highway? Our only answer is that our engineers have made these determinations based on priorities, warrants, money, size of job, and equipment. That is our best defense.

Question

Would a failure to provide inspectors and perform skid testing be proved negligence?

Carlson

No, the failure to inspect is not the basis of liability in California. But, having an inspection system and using it properly is the basis of defending a lawsuit. However, having one does not mean you will win.

Question

If you had a hazardous condition and you failed to do the inspection, would this be a basis for liability?

Carlson

The condition of the highway is the basis of liability, then proving there was notice of the condition. If the condition had been there for months, the passage of time would be a basis of notice and liability.

Question

If research should show that higher skid resistance was needed along express highways at curves or stop signs than on tangents and you corrected this, thereby causing a differential in skid numbers, what would be the legal concerns?

Gartner

That was a topic of extensive discussion brought up at our committee session but the question was not resolved. We would just be moving the danger point from 1 point on the pavement to another. Correcting skid resistance in 1 lane and not correcting it in the adjacent lane can create a worse hazard than by not correcting it in either lane.

Goodwin

I believe the question is: What liability does the engineer have if he or she has increased, in a curve, the minimum skid number but has not increased the value on the road before the curve? In this case, you have 2 different skid numbers, both of which may meet your legal requirements but may require a different driver response.

Carlson

We have had 1 situation in California in which a lane of asphalt was added onto a concrete section. Here, there may be a different coefficient of friction. Test results vary at every test spot. For example, in 1 spot the skid number was 36, in another, 40. You're going to have this variation in highways at different locations. I have not run into a liability situation in which 1 section of road was higher or lower than another adjacent section. I do not see how it adds to or detracts from liability.

Question

Do we need some guidance from the legal profession about whether we should try to establish minimum skid numbers with the best technology we have now?

Carlson

State engineers who are making replies to different agencies should consult with their attorneys because state laws vary, the duty of the engineer varies, the duty of the state varies, the laws of evidence vary from state to state.

Goodwin

Suppose that the Federal Highway Administration imposes on each state a set of minimum skid resistance values. What, then, is the engineer's liability?

Somerville

If the federal government sets standards, someone must provide the money to implement them. The money has to come from somewhere to meet the standards that have been imposed.

Question

I wasn't thinking of having legislators involved in setting standards when I first asked the question. Would we not be better off as individual state highway departments and departments of transportation to set some numbers for our state, for our people's driving habits, rather than have standards imposed on all the states?

Gartner

Is there a "magic number"? I think that there is, but what it is I do not know. I would rather see the setting of minimum skid numbers delayed. But, I think that we should bring all our roads up as high as possible. There is a moral requirement as well as a legal requirement to provide minimum skid resistance.

Loutzenheiser

If you are going to name a number you must have a base level of measurement. We are lacking this at the moment. And, a single number would not provide the necessary flexibility for widely varying conditions.

Question

If a state highway department set its own levels of performance, they would serve as persuasive evidence in the same way as an established safety program would, would they not?

Carlson

If you failed to meet established standards and warrants, that would be used as evidence by the plaintiff. If the standards and warrants were met, that would be used as evidence by the state's attorney.

Question

Would you not be better off in either case?

Goodwin

In other words, would the states not be better off to have an effective highway safety program and establish warrants?

Carlson

Very definitely yes, but they have to have the wherewithal to meet those standards and warrants.

Question

A previous question asked what degree of liability does a highway research engineer have if he or she builds a full-scale pavement test and an accident occurs on that experimental pavement? And it was determined that the engineer might have some liability. If in another state it had been shown that a given method of construction was good in alleviating skid resistance, and to determine whether that method applied in a different environment you ordered a full-scale test to be performed, would the same level of liability exist?

Carlson

Based on California law, yes.

Somerville

I think that warning signs would provide some protection from liability for the engineer.

Question

If there were an accident in a section that was supposed to be resurfaced in the next year, would it be a defense that the state was attempting, within fund limitations, to resurface the worst sections and had not been able to do this section because others were in poorer condition? Is that a defense?

Somerville

The basis for the defense would be that the state budgeted its money on the best engineering advice. The basis for establishing liability would be that the state did not follow the best engineering advice in establishing priorities.

Carlson

It is a defense but not a complete defense. It is just a piece of evidence that the state will introduce to prove that it had a reasonable system of warrants and priorities. But the case will still go to a jury, at least in California, and the jury will determine the reasonableness of your warrants and your priorities. But it is the best evidence in this situation.

Question

In Kansas, we had an improvement under contract that we were not allowed to put in evidence. It was not a matter of whether we were doing the best possible job. It was only a matter of whether the condition was dangerous. Would you care to comment on that?

Carlson

As I indicated earlier all the laws in all the states vary. I hope you follow the warning on the patent medicine bottles that, when the pain persists, consult your lawyer.

PREDICTION OF PAVEMENT SKID RESISTANCE FROM LABORATORY TESTS

W. G. Mullen, North Carolina State University at Raleigh

The objectives of this research were to develop laboratory tests for pre-evaluating aggregates and paving mixtures to predict skid resistance properties in the field and to evaluate field installations for correlation of field and laboratory polishing exposures. A usable correlation was found between British portable tester measurements and field skid-trailer measurements at test speeds of 20, 30, 40, and 50 mph (32.19, 48.28, 64.37, and 80.47 km/h). Different correlations were obtained for open-graded mixes and for dense-surface mixes. Field wear versus laboratory wear correlation was attempted by coring pavements after field testing and then polishing cores to terminal polish in the circular-track machine. The full "as new" polish curve was obtained by remixing and molding unworn portions of the field cores into laboratory specimens for polish in the circular-track machine. New and worn polish curves when compared gave the extent of circular-track wear experienced in the field. Comparisons were valid for that mix only and could not be combined from different mixes to give a traffic exposure versus circular-track equivalency. It was possible, however, to establish an upper limit for field wear equivalent to 3 hours or less of machine wear. The establishment of an upper limit allows prediction from laboratory tests of maximum field polish that may be anticipated for a given pavement mixture design. Examples are given in the report.

•THE laboratory method used for evaluating aggregates and pavement mixes was the North Carolina State University small-wheel circular track, which has been adequately described in other publications (1, 2, 3, 4, 5, 6). The laboratory friction measurement equipment was the British portable tester, and the field friction-measurement equipment was the North Carolina State Highway Commission skid trailer. (NCSHC is now the Department of Transportation and Highway Safety.) Both devices conformed to applicable ASTM standards.

During the research, a field correlation was developed between the NCSHC skid trailer and the British portable tester by concurrent field measurements on a large number of dense-graded and open-graded bituminous pavements. Test speeds for the skid trailer were 20, 30, 40, and 50 mph (32.19, 48.28, 64.37, and 80.47 km/h).

Using cores sampled from the test pavements, age and traffic data, and a remix scheme, we obtained the level of field polish achieved in terms of circular-track exposure and new pavement skid resistance. From this information we were able to develop a method to predict maximum field polish during the service life of a pavement from laboratory tests of the proposed pavement mixture.

FIELD AND LABORATORY FRICTION MEASUREMENTS

The laboratory measurement system used in this research was the British portable tester (ASTM E 303-69). The field measurement method employed by the NCSHC was the skid trailer (ASTM E 274-70).

Publication of this paper sponsored by Committee on Surface Properties-Vehicle Interaction.

We correlated the 2 measurement systems by conducting a series of field tests on I-2 and open-graded mixture pavements with the skid trailer and then the British portable tester in the same skid traces. Skid-trailer test velocities were 20, 30, 40, and 50 mph (32.19, 48.28, 64.37, and 80.47 km/h). Tests were conducted over a period of approximately 8 months beginning in January 1972. At the same time that friction measurements were made, cores 6 in. (152 mm) in diameter were taken from each site, 3 from the inner wheel path and from between wheel paths, to later correlate laboratory circular-track machine wear versus field wear.

Field sites were selected to sample a variety of I-2 mixes in the state, all of the open-graded mixes in service at that time, and as many high-traffic long-service I-2 pavements as could be located. The long-service pavements were selected for terminal or near terminal polish to compare to the circular-track polish results. Eight open-graded mix pavements and 35 I-2 dense-graded mix pavements were tested for the British portable number-skid number (BPN-SN) correlation.

BPN-SN data for each mix at each test velocity were plotted by computer and reduced to a straight-line correlation by using regression analysis. Typical BPN-SN point scatter for the 2 mixes is shown in Figures 1 and 2 for a 40-mph (64.37-km/h) test velocity. Combined BPN-SN-velocity correlations are shown in Figures 3 and 4 without individual points. Statistical data are given in Table 1. BPN and SN values for each site were taken at the same test temperature, but no attempt was made to correct data from different sites to a common temperature. We could not find a temperature correction from published BPN and SN measurement data (1, 7).

CONVERSION NOMOGRAPHS

By using the data from Figures 3 and 4, we developed the BPN-SN-velocity correlation charts shown in Figures 5 and 6. We are not implying that the same correlations would be found for other skid trailers and other British portable testers, but we are fairly confident that these are reasonable relationships for the North Carolina equipment. This is how to use the nomograph in Figure 6 (the same procedure applies to Fig. 5): A circular-track test reveals that the terminal polish value for an I-2 mix is 45. Read from the nomograph in Figure 6 that a field pavement subject to equivalent wear would record SN values of 54, 46, 39, and 36 at 20, 30, 40, and 50 mph (32.19, 48.28, 64.37, and 80.47 km/h) respectively. If the field and laboratory exposures are expected to be the same, then a decision can be made about the suitability of the mix for field use.

FIELD WEAR EQUIVALENCY

Correlation Scheme

The scheme for correlating circular-track machine wear with field exposure involved selecting field pavements in various stages of polish and sampling them by taking sets of 3 cores 6 in. (152 mm) in diameter from the inner wheel path after field skid-trailer and British portable tester measurements had been made. Cores were brought to the laboratory, mounted in the circular-track machine, and polished to determine the additional loss of friction, if any, beyond that already attained under field exposure. Then the cores were removed from the track and softened in the oven; the surface was separated from the lower layers; and the polished surface particles and the cut-edge particles were scraped away leaving an unpolished original pavement mixture. From this material laboratory specimens 6 in. (152 mm) in diameter were molded in sets of 3 to represent the original mix when it was placed in the new pavement. The "new" pavement cores were then polished in the circular-track machine to terminal polish to establish the full polishing curve for the mix. The comparison scheme, in which the 2 curves are plotted together and a circular-track wear-time equivalent to field exposure is read, is shown in Figure 7.

The age, traffic mix, and accumulated wheel passes for each pavement were obtained by calculation from NCSHC records. We did not try to adjust to equivalent wheel passes by assigning weights to various wheel loads. Field exposure was recorded as accumu-

Figure 1. BPN-SN open-graded mix data scatter at 40 mph.

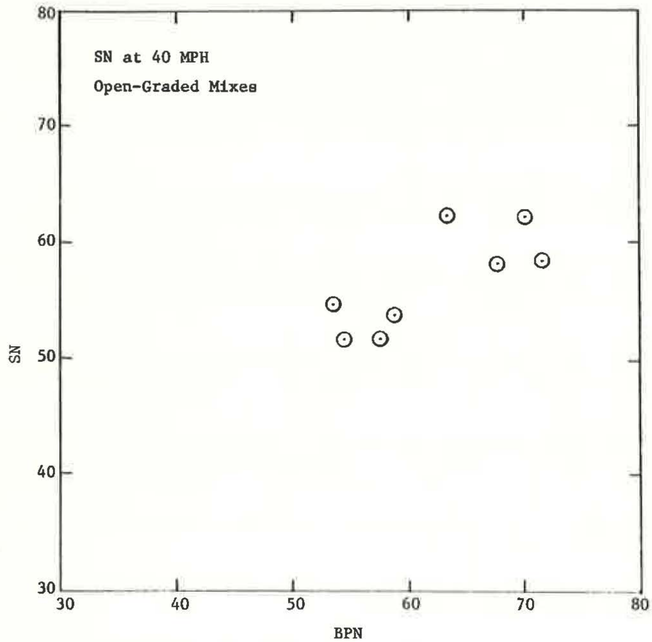


Figure 2. BPN-SN I-2 mix data scatter at 40 mph.

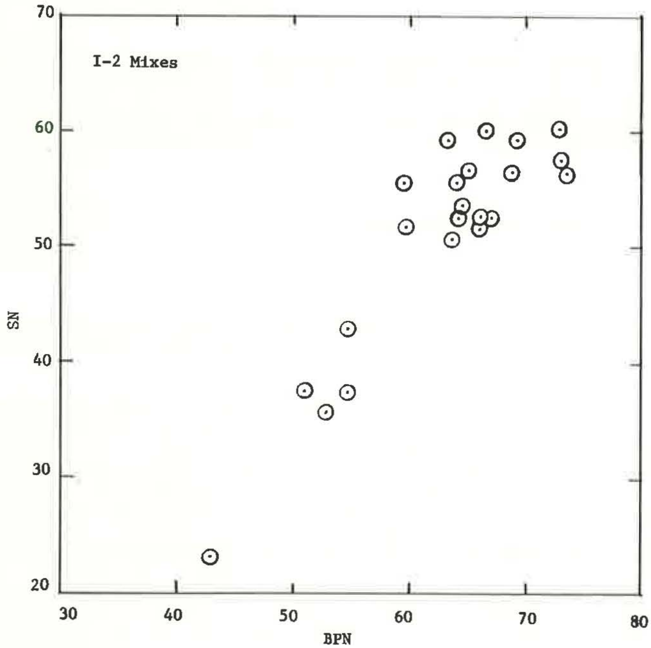


Figure 3. BPN-SN test speed correlations for open-graded mixes.

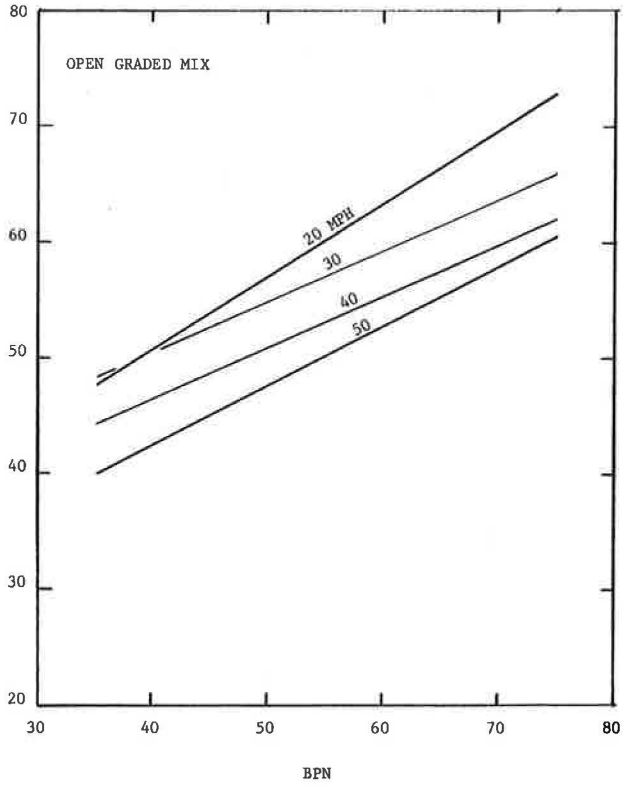


Figure 4. BPN-SN test speed correlations for I-2 mixes.

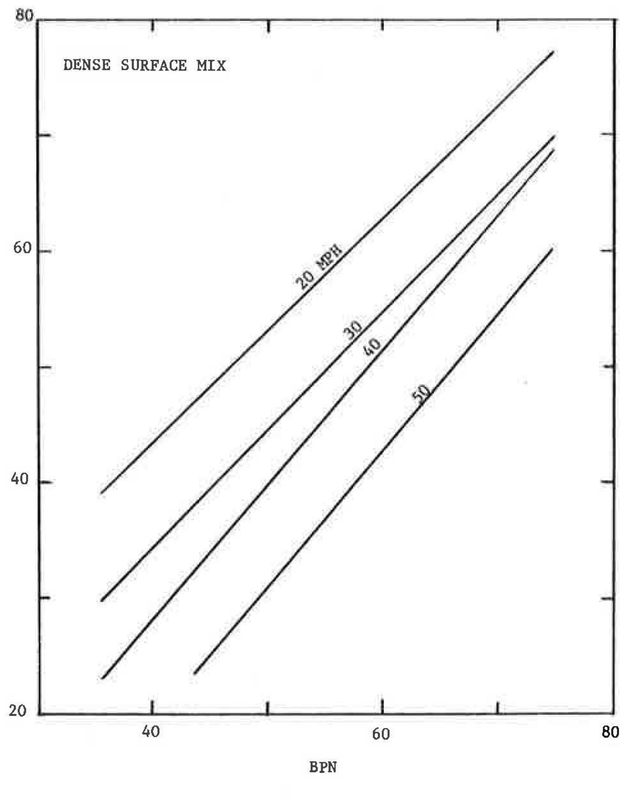


Table 1. BPN-SN speed correlation statistics.

Test Speed (mph)	Mix	Samples	Linear Regression Equation	Correlation Coefficient
20	I-2	22	SN = -4.171 + 0.985 BPN	0.830
30	I-2	22	SN = -6.702 + 1.023 BPN	0.871
40	I-2	22	SN = -19.465 + 1.118 BPN	0.903
50	I-2	22	SN = -28.931 + 1.196 BPN	0.925
20	Open graded	8	SN = 26.232 + 0.621 BPN	0.825
30	Open graded	8	SN = 33.530 + 0.437 BPN	0.629
40	Open graded	8	SN = 28.690 + 0.451 BPN	0.774
50	Open graded	8	SN = 22.254 + 0.512 BPN	0.857

Note: 1 mile = 1.6 km.

Figure 5. BPN-SN-velocity nomograph for open-graded mixes.

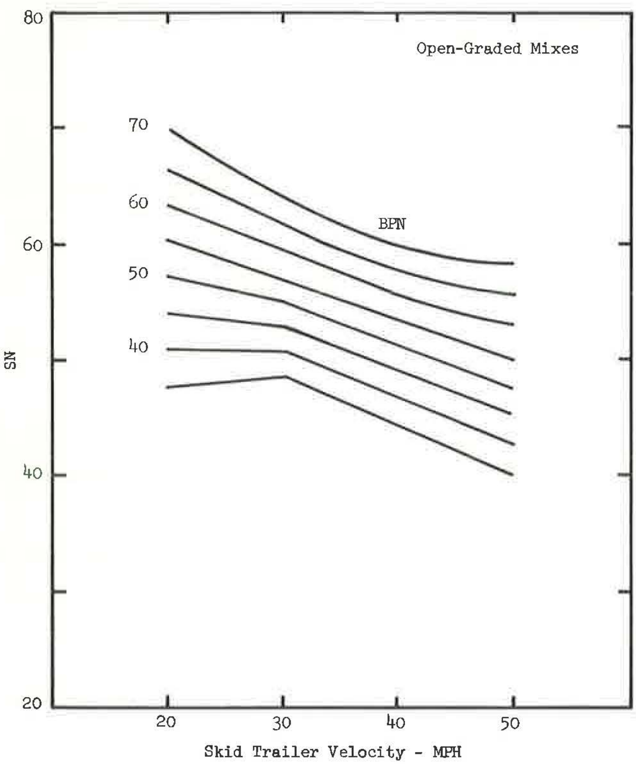


Figure 6. BPN-SN-velocity nomograph for I-2 mixes.

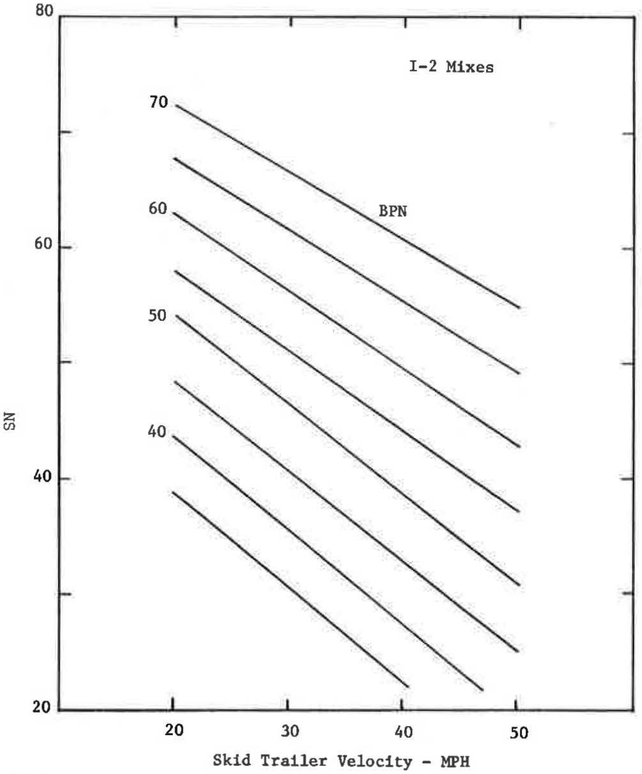
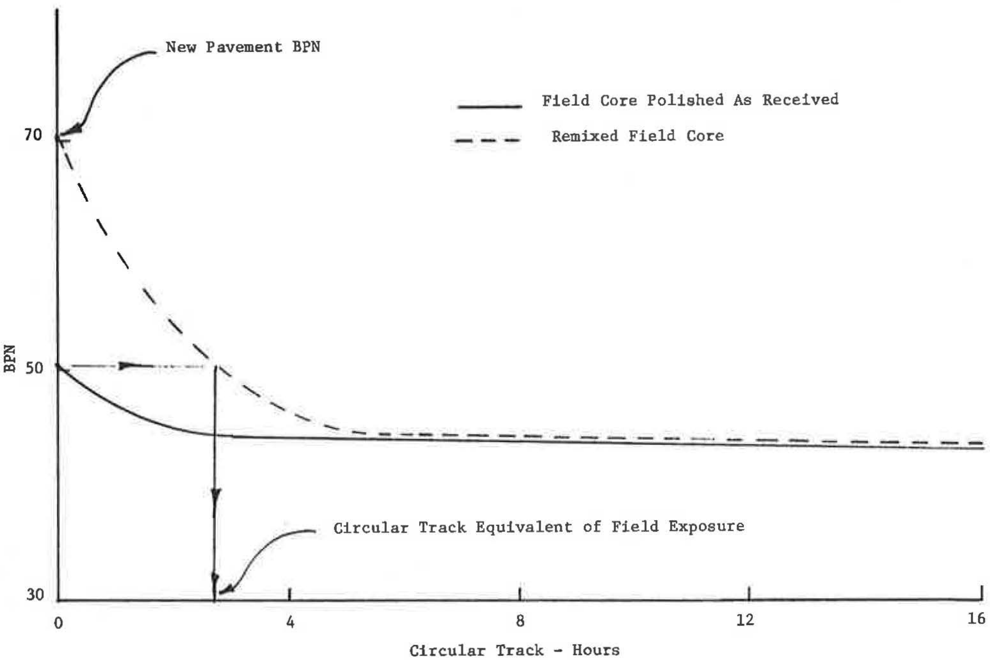


Figure 7. Circular-track versus field exposure equivalency scheme for pavement cores.



lated wheel passes per lane and pavement age in years. Age for I-2 mixes ranged from 0.25 to 11.50 years, and accumulated wheel passes ranged from 300 thousand to 41 million.

In the laboratory wear program, laboratory control specimens were included with each group of pavement cores. For the remix cycle, worn control specimens also were remixed, and a new set of control specimens were included. Original and remix polishing curves for the control specimens were identical for all practical purposes, which indicated that the remix scheme is feasible for approximating original mix properties.

Evaluation of Data

We hoped to be able to state when the work was completed that a direct equivalency between circular-track hours and some function of accumulated wheel passes exists. It is evident from the values for machine hours versus wheel passes obtained by using the scheme in Figure 7 and recorded in Table 2 that no such correlation was obtained for I-2 mixes or for open-graded mixes.

For a given pavement mixture, if a field wear curve could be established, it is expected that a correlation would be obtained for that pavement mixture only. Finding pavements of different ages with identical mix designs and aggregates was unsuccessful mainly because approved aggregates from different sources are substituted in a given production area because of the pressures of supply and demand.

We believe that the most important aspect of the correlation study is that the I-2 mixes had experienced field wear equivalent to less than 2 hours of machine wear in most cases and that the open-graded mixes had experienced in the field less than 3 hours of equivalent machine wear. In other words, mixes in the field that had been exposed to as many as 40 million wheel passes had not polished beyond 3 hours of circular-track machine time. This indicates that the circular-track test is adequate to attain wear at least equivalent to field wear.

Finding an upper limit on field wear with the circular-track method should allow a constructive prediction of field polish limits from laboratory tests. Figure 8, a simple histogram from Table 2 data, shows that 87.5 percent of all I-2 pavements sampled had field polish equivalent to less than 1.5 hours of circular-track exposure and 100 percent had field polish equivalent to less than 2.25 hours of circular-track exposure. For the open-graded mixes, 87.5 percent of the samples showed less then 2.25 hours of equivalent circular-track polish in the field and 100 percent showed less than 3 hours of equivalent circular-track polish in the field.

I-2 mixes sampled went up to 11.5 years in service; the newer open-graded mixes were limited to a maximum 3.67 years in service. It would seem reasonable to estimate that field polish for I-2 mixes would not exceed the 96 percent value of 1.75 hours of circular-track exposure. But, in view of the limited field service of the open-graded mixes, it would seem prudent to estimate that field polish might reach the 100 percent value of 3.00 hours of circular-track exposure.

Prediction of Maximum Field Polish

Examples of maximum field polish prediction from laboratory control mixes plotted in Figure 9 and the nomographs from Figures 5 and 6 replotted for convenience in Figures 10 and 11 respectively are as follows:

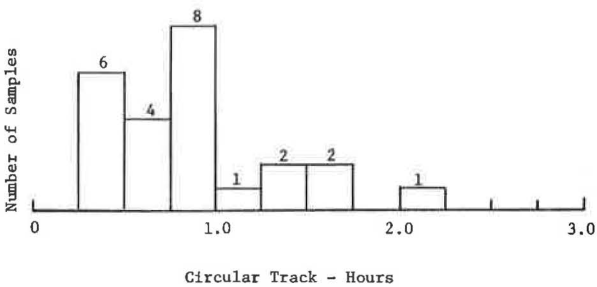
1. For open-graded mixes enter Figure 9 at 3.0 hours on the abscissa, intersect the open-graded mix curve, and pick off BPN of 47 from the ordinate. Enter Figure 10 with BPN of 47, draw the intermediate BPN curve, and pick off predicted SN values for velocity values as follows: (a) 20 mph, 55 SN; (b) 30 mph, 53 SN; (c) 40 mph, 50 SN; and (d) 50 mph, 46 SN. Compare predicted SN values to criteria acceptable for field use.
2. For I-2 mixes enter Figure 9 at 1.75 hours on the abscissa, intersect the I-2 mix curve, and pick off BPN of 52 from the ordinate. Enter Figure 11 with BPN of 52, draw intermediate BPN curve, and pick off predicted SN values for velocity values as follows: (a) 20 mph, 56 SN; (b) 30 mph, 48 SN; (c) 40 mph, 41 SN; and (d) 50 mph, 34 SN.

Predicting field SN values from circular-track results is a practical procedure that can be applied to any laboratory mix before it is chosen for field construction.

Table 2. Field exposure, circular-track data for I-2 and open-graded mix pavements.

Mix	Route	County	Age (years)	Tire Passes (thousands)	Equivalent Circular-Track Hours
I-2	US-64E	Wake	6.42	17,877	0.60
I-2	US-70E1R	Wake	11.50	41,230	0.82
I-2	US-70E1L	Wake	11.50	20,231	1.00
I-2	US-70E2R	Wake	9.75	22,444	1.00
I-2	US-70E2L	Wake	9.75	10,655	0.92
I-2	US-70E3R	Wake	4.92	12,756	0.30
I-2	US-70E3L	Wake	4.92	6,063	0.55
I-2	US-64E	Randolph	0.33	531	0.70
I-2	NC-24	Onslow	6.16	38,376	0.50
I-2	NC-24	Cumberland	8.67	20,155	1.30
I-2	US-17	New Hanover	7.00	10,117	1.00
I-2	US-258-1	Lenoir	7.92	12,922	1.35
I-2	US-258-2	Onslow	10.08	17,971	1.20
I-2	US-258-3	Onslow	10.08	27,138	1.70
I-2	US-74E	Union	5.16	19,565	0.85
I-2	US-74E	Mecklenberg	5.33	20,720	0.35
I-2	I-95 Conn.	Nash	4.58	12,606	0.35
I-2	NC-343	Camden	0.42	316	2.20
I-2	I-85	Alamance	9.67	44,245	0.70
I-2	US-13	Gates	0.25	380	0.45
I-2	US-17	Perquimmons	0.25	519	0.50
I-2	US-301	Nash	2.50	28,758	0.85
I-2	US-220L	Randolph	1.50	1,479	0.70
I-2	US-220R	Randolph	1.50	3,350	1.65
I-2	US-158	Camden	0.25	793	0.90
Open graded	US-70ER	Buncombe	1.67	4,884	0.90
Open graded	US-70EL	Buncombe	1.67	2,233	1.30
Open graded	NC-152	Rowan	2.75	1,967	0.55
Open graded	US-1	Lee	2.58	3,301	0.60
Open graded	US-401	Cumberland	0.67	637	2.90
Open graded	NC-58	Greene	3.67	2,648	2.05
Open graded	US-220	Guilford	2.00	7,380	1.00
Open graded	NC-191	Buncombe	1.83	255	0.25

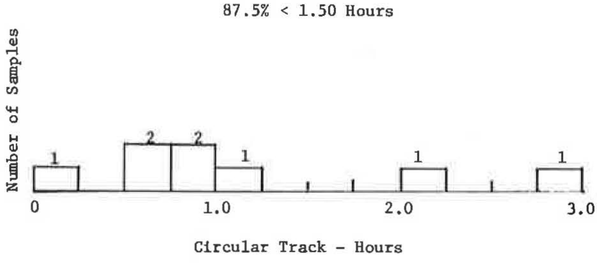
Figure 8. Field polish versus circular-track polish histogram.



I-2 Mixes: 100% < 2.25 Hours

96% < 1.75 Hours

87.5% < 1.50 Hours



Open-Graded Mixes: 100% < 3.00 Hours

87.5% < 2.25 Hours

75% < 1.50 Hours

Figure 9. BPN versus test duration for I-2, open-graded, and No. 4 mixes.

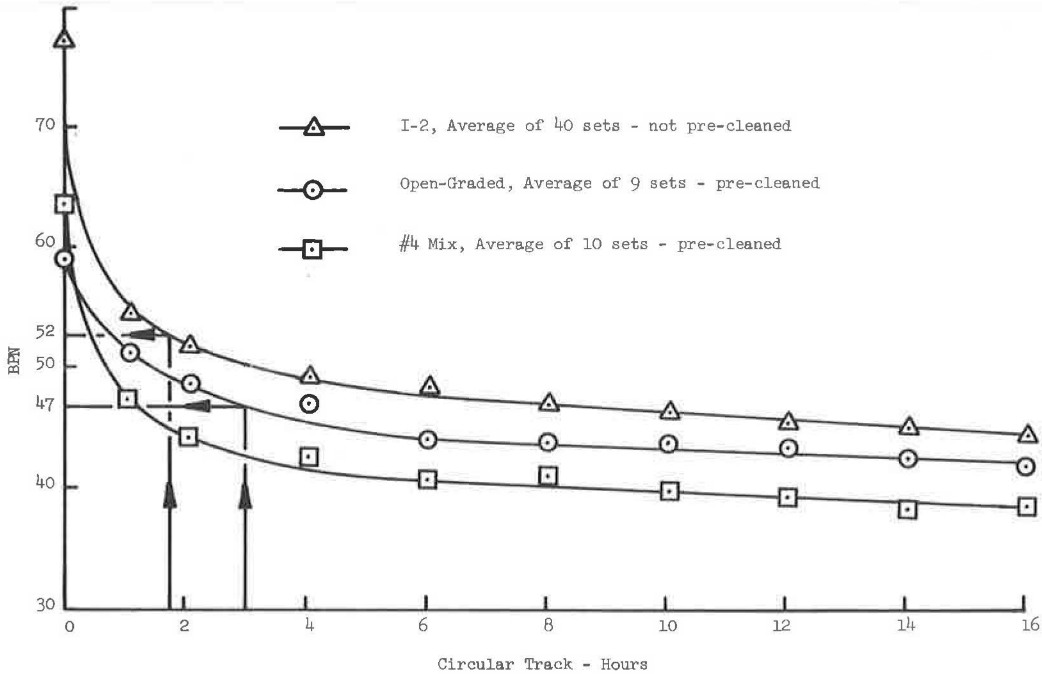


Figure 10. BPN-SN-velocity conversion example for open-graded mix.

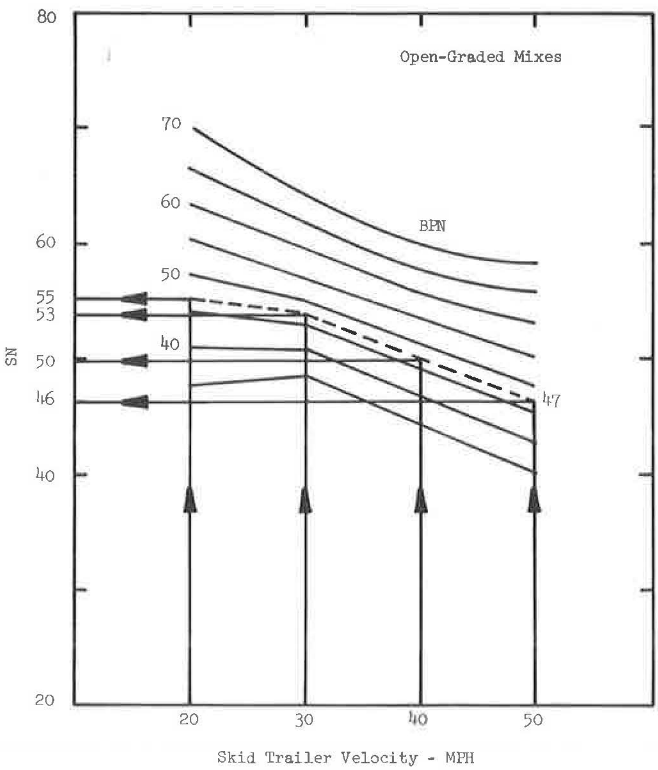
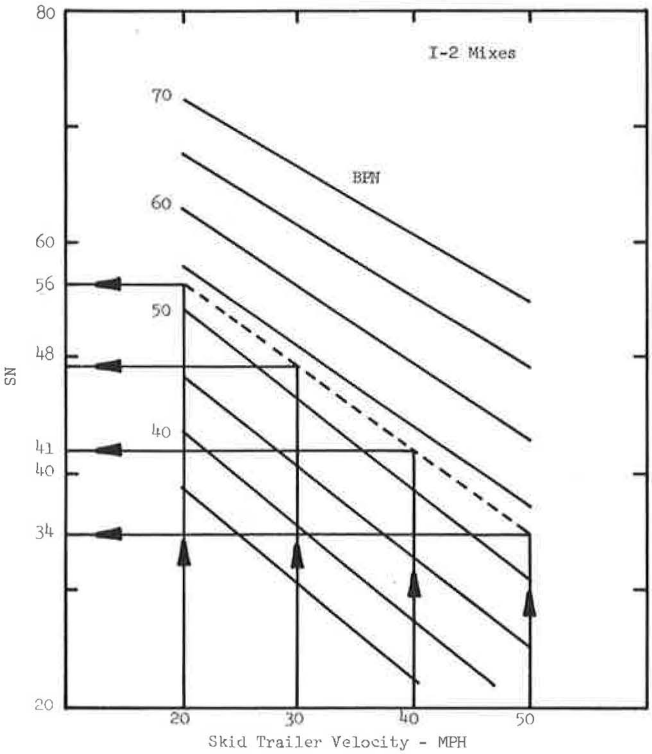


Figure 11. BPN-SN-velocity conversion example for I-2 mix.



Test Time Reduction

The finding from field and laboratory measurements that field polish for the pavements investigated never exceeded 3.0 hours of circular-track exposure implies that the circular-track exposure necessary to establish the polish curve for any bituminous mix can be limited to a maximum of 8 hours and possibly could be reduced to 6 hours. A 6-hour actual exposure time would allow completion of a mixture evaluation in 1 working day after sample preparation and mounting.

SUMMARY

It has been possible during the course of this research to develop methods for determining the wear and polishing properties of aggregates in the laboratory and to predict with reasonable assurance the limits on field polishing of aggregates and mixtures based on laboratory tests.

Field correlation was established between the British portable tester friction measurement method used both in the laboratory and in the field and the NCSHC skid trailer. This correlation allows translation of laboratory friction measurements into skid-trailer SNs at velocities from 20 to 50 mph (32.19 to 80.47 km/h).

From field-laboratory wear correlation studies a method was developed whereby an upper limit on field polish can be predicted for I-2 mixes and open-graded mixes based on circular-track polish results. This prediction method allows the preevaluation of mixes for field polish resistance adequacy before construction is undertaken.

ACKNOWLEDGMENTS

The work reported in this paper was sponsored by the North Carolina State Highway Commission in cooperation with the Federal Highway Administration. The work was performed by the author in the Civil Engineering Laboratories of North Carolina State

University. In addition, progress of the research has been aided by many of the personnel of the North Carolina State Highway Commission and the Federal Highway Administration as well as by several members of the North Carolina State University faculty, staff, and student body. The opinions, findings, and conclusions reported in this paper are those of the author and not necessarily those of the sponsors or the Federal Highway Administration.

REFERENCES

1. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures, Final Report. North Carolina State University at Raleigh, Highway Research Program, Proj. ERD 110-69-1, Sept. 1972.
2. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures, Interim Report. North Carolina State University at Raleigh, Highway Research Program, Proj. ERD 110-69-1, June 1970.
3. Mullen, W. G., Dahir, S. H. M., and Barnes, B. D. Two Laboratory Methods for Evaluating Skid-Resistance Properties of Aggregates. Highway Research Record 376, 1971, pp. 123-135.
4. Dahir, S. H. M., and Mullen, W. G. Factors Influencing Aggregate Skid-Resistance Properties. Highway Research Record 376, 1971, pp. 135-148.
5. Dahir, S. H. M. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures. North Carolina State University at Raleigh, PhD dissertation, 1970.
6. Barnes, B. D. The Development of a Wear and Polish Machine for Laboratory Evaluation of Skid Resistance Properties of Paving Mixtures. North Carolina State University at Raleigh, unpublished senior thesis, 1970.
7. Selected Annotated References on Skid Resistance. North Carolina State University at Raleigh, Highway Research Program, Proj. ERD 110-69-1, Oct. 1968.
8. Selected Annotated References on Skid Resistance, Supplement. North Carolina State University at Raleigh, Highway Research Program, Proj. ERD 110-69-1, June 1971.

DISCUSSION

Charles R. Marek, University of Illinois

The following discusses the relationships between skid-trailer numbers and British portable tester numbers for 4 types of pavement surfaces in Illinois. The data developed in Illinois support the relationships reported by Mullen. Mullen's correlations (as established in North Carolina) are not exactly the same as those developed from the Illinois research project. However, similarities exist, and the differences probably can be attributed to slight differences in state skid trailers and test procedures.

During the summer of 1971, extensive field work was performed by personnel of the University of Illinois in cooperation with research personnel from the Ottawa Physical Research Laboratory of the Illinois Department of Transportation. Approximately 50 sites were selected and studied. Field measurements taken at each site included, in part, state skid-trailer skid resistance values [at 40 mph (64.35 km/h)] and British portable tester skid resistance values. Four types of pavement surfaces, Illinois Class I, Class B, and Class A bituminous surfaces and portland cement concrete (PCC) surfaces, were studied. Skid-trailer tests were followed immediately by tests in the same skid traces with the British portable tester (9).

As a result of the data collection effort expended on Illinois Highway Research Project 406, limited data were obtained that permitted correlation of the BPNs with the Illinois skid-trailer values. To establish the correlation, a reduced major axis technique was employed to construct the line that best approximated or "fit" the observed trend. (This line minimizes the sum of the areas of the triangles formed by lines drawn from each point to the desired line and parallel with the x and y axes.) The 2 skid resistance variables for a specific surface type are plotted in Figures 12, 13, 14, and 15. For each surface type, the best fitting line and the correlation coefficient were established. This line has been drawn on each graph. All pertinent information is given in Table 3.

The information in Table 3 shows that a good correlation resulted for the 2 methods of determining skid resistance coefficients on bituminous surfaces (regression coefficient > 0.90). Greater variability was experienced on PCC pavement surfaces (regression coefficient $\simeq$ 0.80). The relations in Figures 12, 13, 14, and 15 indicate that the British portable tester is responsive to measuring the coefficient of friction of pavement surfaces.

Mullen's approach to correlation of laboratory wear with field exposure appears promising. Further, the finding that the wear exhibited in the laboratory, after only several hours of exposure, was more extensive than that produced in the field after 40 million wheel passes is significant. Preevaluation of material performance can now be made based on sound technical data. This can result in more effective use of materials and in the construction of pavement surfaces that will exhibit desirable skid resistance qualities throughout their design life.

REFERENCE

9. Marek, C. R. Review, Selection, and Calibration of Accelerated Wear and Skid Resistance Testing Equipment, Interim Report, Phase 1. University of Illinois, Transportation Engineering Series No. 2, ICHRP Series No. 134, Nov. 1972.

S. Dahir, Pennsylvania State University

Mullen's paper is of particular interest to us at Pennsylvania State University because it is similar to some of our research findings. The basic objective of our research has been to preevaluate in the laboratory bituminous pavement mixtures and aggregates to predict skid resistance levels for the same mixtures and aggregates in the field. We have attempted to obtain a correlation between skid-trailer skid numbers and BPNs and have both procedures conform to respective ASTM methods. Taking measurements with both instruments on the same pavement at the same time was contemplated but not done because of constraints on time and help.

We attempted some correlations between SN_{40}s taken on Pennsylvania ID-2A dense mixes of 11 Pennsylvania test strips and BPNs taken on cores obtained from the same test sections and polished in the laboratory. The correlation lines, in general, resembled but differed from the BPN-SN_{40} correlation reported by Mullen for the North Carolina I-2 dense mixes.

In our work, we compared BPNs for cores polished with a flat 3.5- by 5.5-in. rubber pad in a reciprocating machine to SN_{40}s taken during 4 years of pavement polishing by traffic and corrected for temperature by PennDOT. Pavement cores terminally polished in the laboratory with a rubber surface were compared to pavements in the field polished with the passage of millions of vehicles over a 4-year period. We wanted to know how valid the results of our prediction curves would be if we used a laboratory polishing method to predict field skid resistance. When we saw Mullen's results obtained by simultaneous SN_{40}-BPN measurement correlations, we gained a greater measure of confidence in our laboratory polishing procedure as a predictor of SN_{40} after the pavement had been exposed to several million vehicle passes. Figures 16 through 19 show some of our results.

One thing that bothers us is the scatter of data around the average best fit curves. This scatter, both in our data and in Mullen's data (Fig. 2), shows a variation of approximately 6 ± 2 SN_{40} for a confidence interval covering 95 percent of the data points. To obtain a safe prediction, then, 1 of 2 approaches may be used. The lower confidence interval value may be used to predict about 6 (or more) skid numbers lower than the expected average, resulting in a satisfactory design for poorly performing aggregates and mixes but in overdesign for the good performers (as many as 12 or more SN_{40}s higher than may be required). Or, curves parallel to the average curve may be drawn

Figure 12. Relationship between BPN and state skid-trailer number for Class I surfaces.

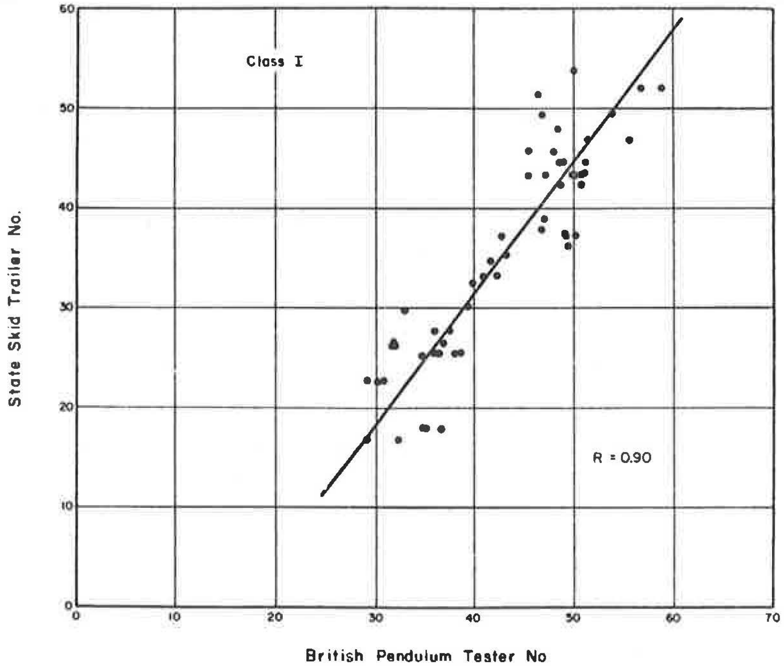


Figure 13. Relationship between BPN and state skid-trailer number for Class B surfaces.

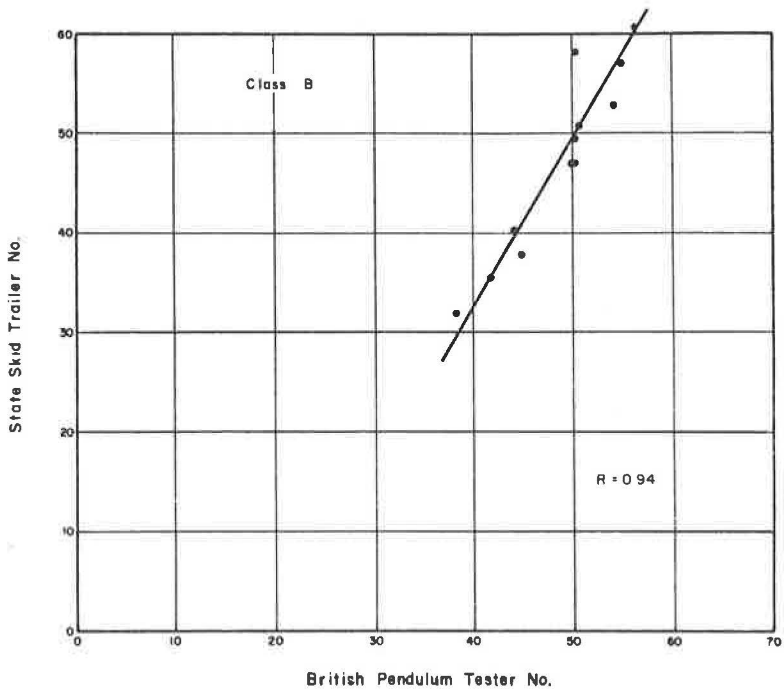


Figure 14. Relationship between BPN and state skid-trailer number for Class A surfaces.

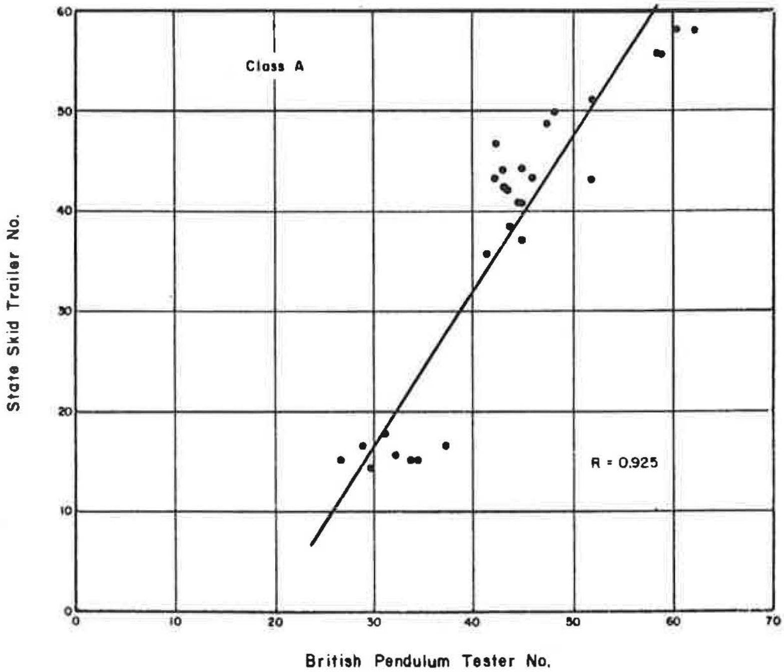


Figure 15. Relationship between BPN and state skid-trailer number for PCC surfaces.

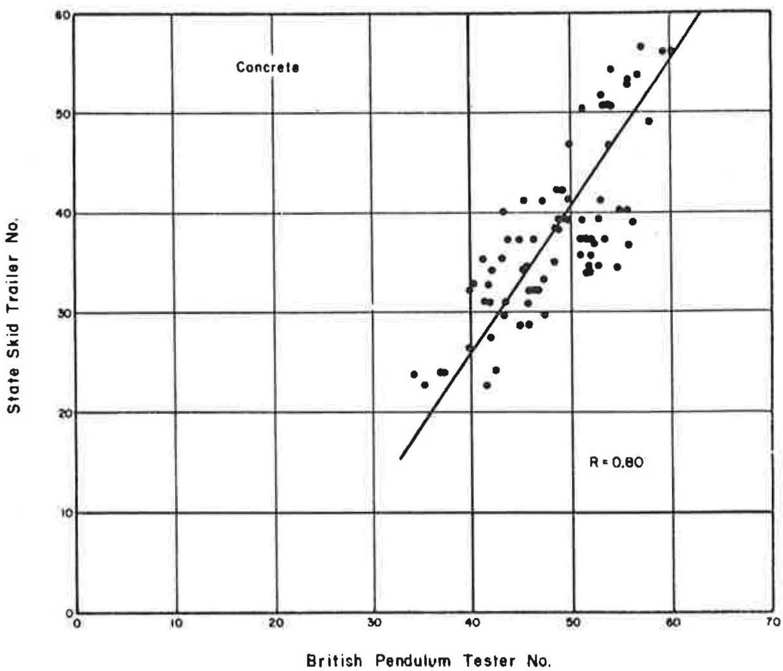


Table 3. Results from statistical analysis.

Type of Surface	N	Slope[a]	Intercept[a]	Correlation Coefficient[a]	Slope x on y[b]	Slope y on x[c]
PCC	76	1.4555	-32.477	0.80	1.8264	1.1599
Class I	56	1.3326	-21.769	0.90	1.4750	1.2040
Class B	12	1.6944	-35.366	0.94	1.7958	1.5986
Class A	28	1.5690	-30.538	0.92	1.6955	1.4520

[a]Reduced major axis. [b]Regression of x on y. [c]Regression of y on x.

Figure 16. Lowest 1972 SN_{40} on 44 sections in Montgomery County, Pennsylvania, 4 years after construction versus BPN of laboratory-polished cores.

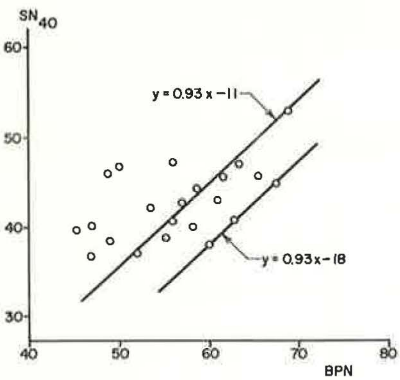


Figure 17. Four-year averages of annual SN_{40} lows for dense mixes containing 9 aggregates versus BPN of laboratory-polished cores from same mixes.

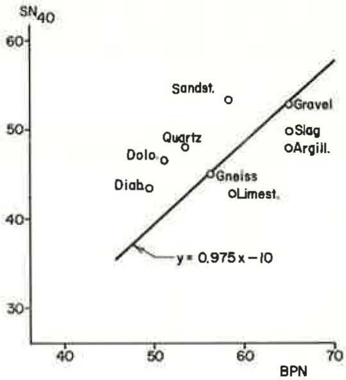


Figure 18. SN_{40} for dense mixes containing 9 aggregates measured 4 years after construction versus BPN of laboratory-polished cores.

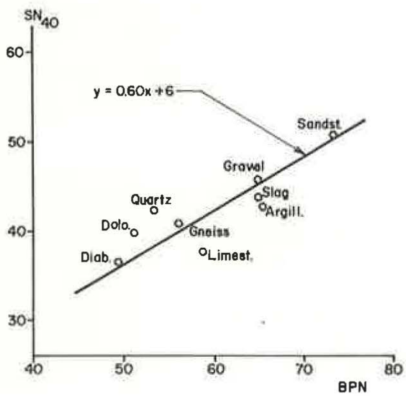


Figure 19. SN_{40} versus BPN for Pennsylvania and North Carolina data of bituminous dense-mix pavements.

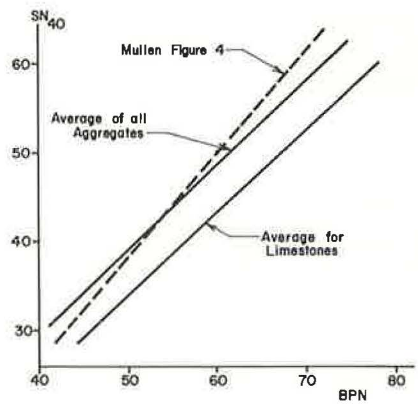
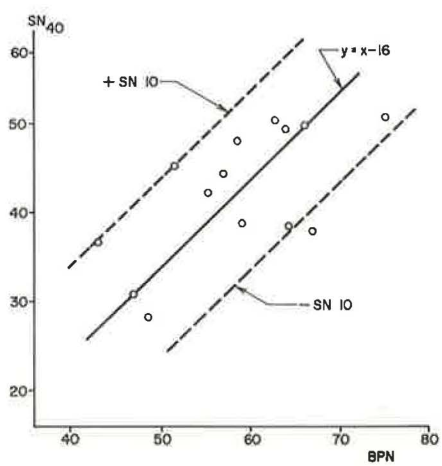


Figure 20. 1972 SN_{40} lows versus BPN of laboratory-polished cores of Pennsylvania limestone test sections.



through data points representing a particular aggregate and mix, and these curves may be used for the types of mixes and aggregates they represent.

The latter may be the more economical of the 2 methods and the more equitable approach when varying quality aggregates of the same type or group are involved, such as in the case of various limestones as may be seen from Figure 20.

AUTHOR'S CLOSURE

The discussions by Dahir and Marek, giving data from Pennsylvania and Illinois, seem to support the findings of the North Carolina BPN-SN correlation work. Differences in the actual correlations obtained are probably attributable to differences in the British portable testers and the skid trailers that were used plus differences in design and construction of the pavement surfaces that were evaluated.

The work presented by the discussants encourages belief in the conclusion that accelerated laboratory test results may be used to predict field performance of mixes and aggregate combinations for wear and polish resistance.

LABORATORY EVALUATION OF AGGREGATES, AGGREGATE BLENDS, AND BITUMINOUS MIXES FOR POLISH RESISTANCE

W. G. Mullen, North Carolina State University at Raleigh;
S. H. M. Dahir, Pennsylvania State University, Middletown; and
N. F. El Madani, Consulting Engineer, Libya

The objectives of this research were to develop laboratory tests for preevaluating aggregates and paving mixtures to predict skid resistance properties in the field and to identify mixture designs incorporating optimum skid resistance and polish resistance. Laboratory evaluation methods developed include a circular-track machine using small-diameter pneumatic tires and a petrographic method for evaluating polish susceptibility based on percentage of hard mineral found in aggregate thin sections. Evaluation of the 2 test procedures is provided, and findings are given of laboratory tests on polish resistance properties of aggregates, aggregate blends, and bituminous mixes.

•USE of the North Carolina State University small-wheel circular track for laboratory studies of aggregate and pavement polishing properties was reported to the Highway Research Board in 1970 by Mullen, Dahir, and Barnes (1). Since that time improvements have been made to the track and studies have been extended to include additional aggregates, variations in bituminous pavement mixtures, and aggregate blends including some with crushed waste glass. The track was turned over to the North Carolina State Highway Commission (NCSHC is now the North Carolina Department of Transportation and Highway Safety) in 1973 for a program of experimental preevaluation of bituminous pavement mix designs.

The major improvement to the track was the substitution of a tire 5.25 in. (133 mm) wide for the previous tire, which was 3.50 in. (90 mm) wide, to widen the wear path and eliminate possible edge effects on measurements made with the British portable tester (ASTM E 303-69). New calibration curves were developed by comparing the laboratory control aggregate results to earlier results with narrower tires. Results from old and new tires were comparable.

Characteristics of the polishing curve from the first hour up to 100 hours were established to help interpret results of polish tests. The petrographic method of comparing the effect of hard and soft mineral content of aggregate particles on polish resistance was extended to include vesicular lightweight aggregates.

Findings of the laboratory study indicated that the petrographic method can predict polishing characteristics of aggregates, that mixture design affects pavement polish level, and that blending of aggregates produces polish levels that reflect an averaging of individual aggregate polishing resistance in proportion to percentages used in the mixtures. Also, it was found that coarse, aggregate-sized crushed glass particles produce mixes with low polish resistance. Mixes containing sand-sized crushed glass particles compare favorably to sand-asphalt mixes for polish resistance.

Publication of this paper sponsored by Committee on Surface Properties-Vehicle Interaction.

PETROGRAPHIC METHOD EXTENSION

Results of the petrographic thin-section method for preevaluation of aggregates reported in 1970 (2, 3) have been extended to include lightweight vesicular aggregates. Three such aggregates produced and marketed in North Carolina that have proven to be relatively polish resistant were included in the study. These aggregates are expanded slates produced by belt sintering or by rotary kiln processes.

Determining the percent of hard minerals in the expanded aggregate was accomplished by assuming that the sintered parts were greater than H = 5 and that the void spaces of the vesicules were less than H = 5, which of course is true. The percent of hard mineral in the expanded aggregate was computed by taking the ratio of the specific gravity of the expanded material to the specific gravity of the unprocessed slate and multiplying by 100.

British portable number (BPN) values for 16-hour circular-track exposure for all the aggregates evaluated, including the 3 lightweight aggregates, are given in Table 1 together with hard mineral percentages. BPN and hardness values for all aggregates also are plotted in Figure 1. Aggregates with hard mineral percentage in the 40 to 70 percent range were more polish resistant than were aggregates of either greater or lesser hard mineral content.

Including the 3 lightweight aggregates seemed to improve the correlation reported previously by filling out the data gap between 40 and 70 percent hard mineral aggregates. The better polish resistance in this hard mineral range resulted because these aggregates are essentially sacrificial, that is, the soft matrix releases the hard grains before they can be polished.

It will be seen later that hard aggregates and soft aggregates do not polish at the same rate. Therefore, the low laboratory friction values for some hard aggregates that are not prepolished by nature or manufacturing process may not be reached in all field exposures.

WIDE TIRE CALIBRATION

For the 1970 reports (1, 2, 3), all polishing data reported were for open-graded mixtures polished with the narrow pneumatic tires. When the wider tires were substituted, it was necessary to develop a new standard aggregate calibration curve. Also, work was begun at this time with dense-graded mixes and with mixes incorporating coarse aggregates of different maximum sizes.

Calibration curves for the standard laboratory aggregate open-graded mix polished with narrow pneumatic tires and the same mix polished with wide pneumatic tires had no significant difference in rate of polishing. In these tests the specimens first were cleaned of surface coating by wiping with solvent to expose aggregates before polishing. For later work this practice was discontinued, and, as a result, initial BPN values were higher. Circular-track polishing without precleaning is more indicative of the course of field polishing even though terminal BPN values do not seem to be affected.

ONE-HOUR CURVE

Other work was done to establish the early part of the polishing curve on the circular track with specimens that were not precleaned with solvent to remove asphalt coating from the aggregate. The polishing curve for the open-graded mix for the first hour is plotted on an expanded horizontal scale in Figure 2 together with the full 16-hour curve plotted on the normally used scale. Loss of 8 to 10 BPNs in the first 2 minutes of exposure probably corresponded to the wearing away of asphalt coating and surface fines in the asphalt coating. Comparison of initial BPN values for specimens precleaned to specimens not precleaned revealed a reduction of about 10 due to precleaning. Also, initial BPN values for specimens not precleaned fluctuated over a wider range than did initial BPN values for those that were precleaned.

CALIBRATION FOR DENSE MIXES

Most of the work done after 1970 was with dense-graded mixes and pavement cores.

Table 1. BPN and range of hard mineral content.

Aggregate Symbol	BPN After 16-Hour Circular-Track Exposure	Percent Mineral Content[a]	
		H = 2 to 4	H = 5 to 7.5
SS-1	58.5	30 to 40	60 to 70
SO-1	54.0	40 to 46	54 to 60[b]
SL-2	46.0	50 to 55	45 to 50
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	55 to 65	35 to 45
LS-3	40.0	65 to 70	30 to 35
LS-2	39.0	95 to 97	3 to 5
LS-1	35.0	93 to 95	5 to 7
TU-1	52.7	33 to 38	62 to 67[b]
ST-1	51.6	31 to 35	65 to 69[b]

[a]Determined by petrographic thin section examination.
[b]Expanded slate lightweight aggregates.

Figure 1. BPN values versus hard mineral content.

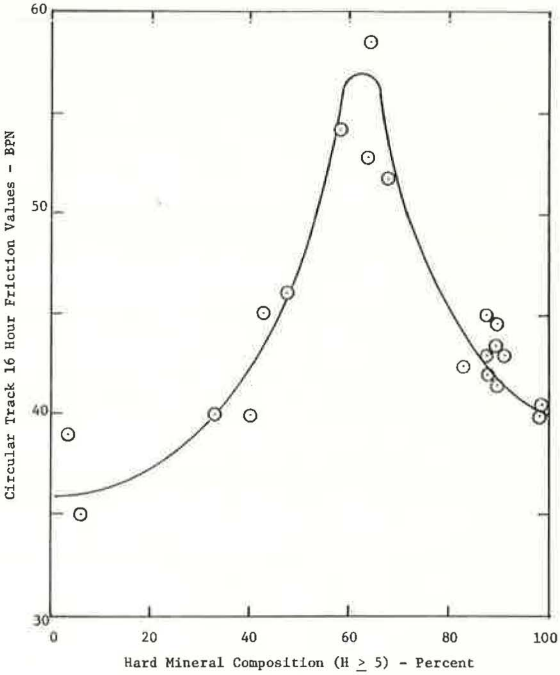
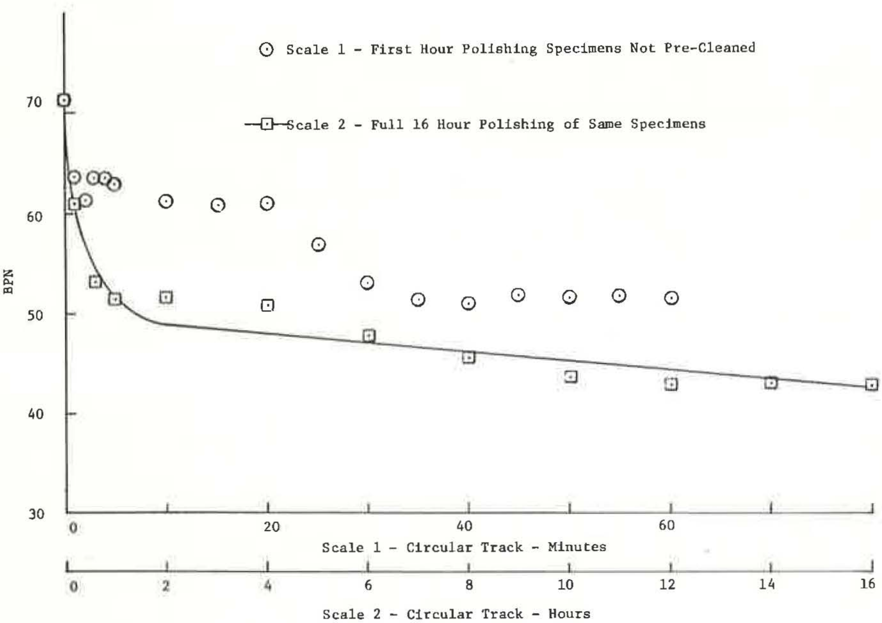


Figure 2. First hour of polishing curve, open-graded mix control aggregate.



The 2 dense-graded mixes used were the No. 4 mix and the I-2 mix having 90 to 100 percent passing the ¾-in. (19.0-mm) and ⅜-in. (9.5-mm) sieves respectively. To establish some control for these mixes, both open-graded and dense-graded laboratory control aggregate mixes were included in early runs to compare for calibration. Comparison curves are shown for these mixes in Figure 3. Open-graded and No. 4 aggregate dense-graded specimens were precleaned with solvent before testing; I-2 dense-graded specimens were not precleaned, which explains to some degree the much higher initial BPN value for the I-2 mix in Figure 3.

It is significant that the I-2 dense-graded mix with the same size stone as the open-graded mix showed higher polish values throughout than did the open-graded mix. When a larger aggregate No. 4 mix was checked, the calibration curve fell below both open-graded and I-2 mix values.

EXTENDED POLISHING

Extended polishing to 100 hours was conducted in a series of tests to determine whether additional exposure would produce significant changes in rate of polishing or level of polishing. Data for 2 sets of I-2 samples are shown in Figure 4. Four other sets representing other aggregates were tested with essentially the same results. About 5 BPNs were lost after 16 hours when exposure was extended to 100 hours and essentially no additional polish was gained after about 30 hours.

In light of other work with field cores (4), we believe that extended polishing on the circular track is of no value in estimating field performance.

NORMAL ASPHALT CONTENT VARIATION

One further series of tests was performed to determine sensitivity of the circular-track procedure to reasonably normal changes in asphalt content (AC) in dense-graded mixtures. Results are given in Table 2 and indicate no particular sensitivity for the range of ACs investigated. None of the specimens showed evidence of bleeding or flushing during the test exposure.

AGGREGATE BLENDING

The petrographic method of aggregate evaluation for laboratory polish resistance (Fig. 1) indicated that polish resistance measured by the circular-track method was highest when an optimum percentage of hard and soft minerals was present within individual aggregate particles. It has been conjectured that this principle could be extended to mixes so that the same benefits might be attained through blending of hard aggregates and soft aggregates in some optimum proportions.

Limestone and Silica Gravel

The results of the limestone and crushed-silica-gravel blend are shown in Figure 5 and indicate that the polishing properties of the blends of these 2 coarse aggregates in an open-graded mix were affected generally in proportion to the percentages of the 2 aggregates in the blend. Each point represents the average for 3 specimens run concurrently. Values are adjusted by the control aggregate. Initial values are for specimens with surfaces precleaned of asphalt coating.

Crushed Glass Blends

A series of test blend mixes were made with crushed waste glass and coarse and fine aggregates with various substitutions of coarse and fine aggregates from limestone, granite gneiss, and natural silica sand sources. The purpose of these trials was to determine the beneficial effects, if any, of blending a waste material with a low polish-resistant aggregate. The mixture used for this investigation was the I-2 mix and 1 mix using only fine aggregates. Mix combinations are given in Table 3. Glass aggregates were obtained by crushing and sizing washed glass bottles obtained from a local glass collection drive. Bottles were put in sacks and smashed with a hammer; the fragments

Figure 3. Wide pneumatic tire correlation curves for the standard aggregate.

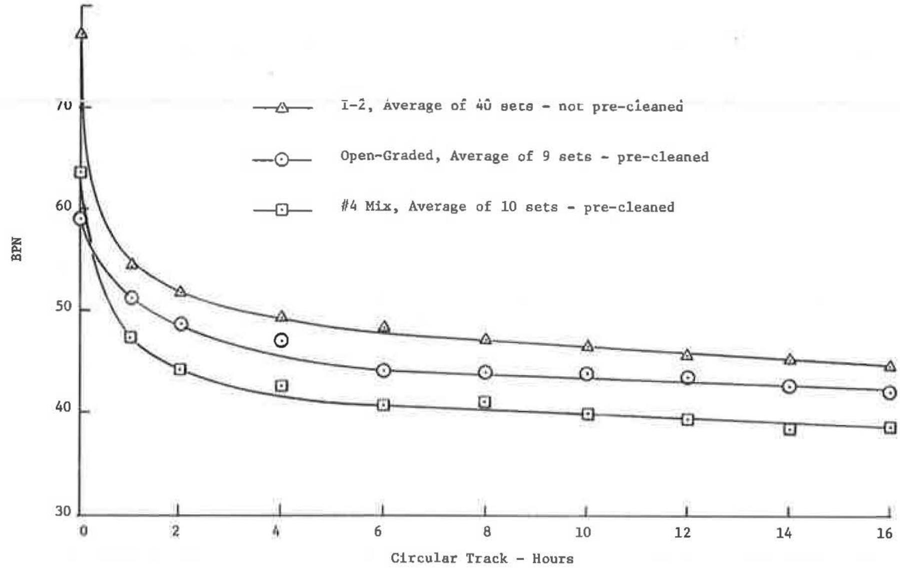


Table 2. Effect of normal range variation of asphalt content on circular-track polish values for control aggregate I-2 mix.

Circular-Track Hours	BPN Versus AC[a]			
	6 Percent	6.5 Percent[b]	7.0 Percent	7.5 Percent
0	76.7	76.3	76.3	76.7
1	50.3	49.3	49.7	49.7
2	52.0	51.7	51.7	51.7
4	50.0	51.7	51.3	51.7
6	48.3	49.3	47.0	48.7
8	48.0	47.7	47.0	47.7
10	47.0	47.0	47.0	46.3
14	45.7	47.0	46.0	43.5
14	43.7	43.7	45.7	42.3
16	43.7	44.0	44.3	42.7

[a]All values adjusted to standard I-2 curve.
[b]AC for normal control mix.

Figure 4. Extended polishing, I-2 standard aggregate control mix.

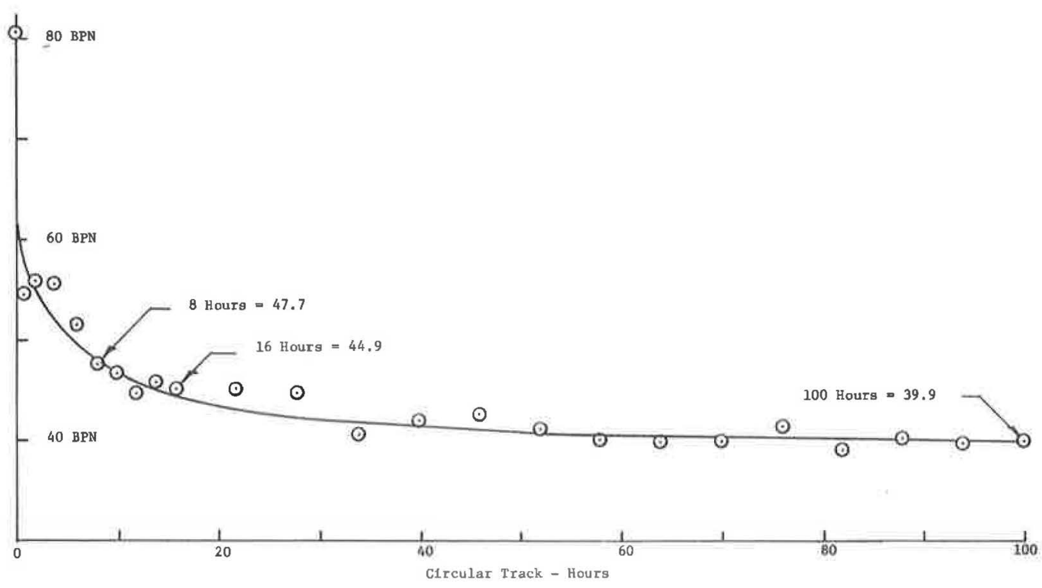


Figure 5. Polishing properties of blended aggregates in an open-graded mix.

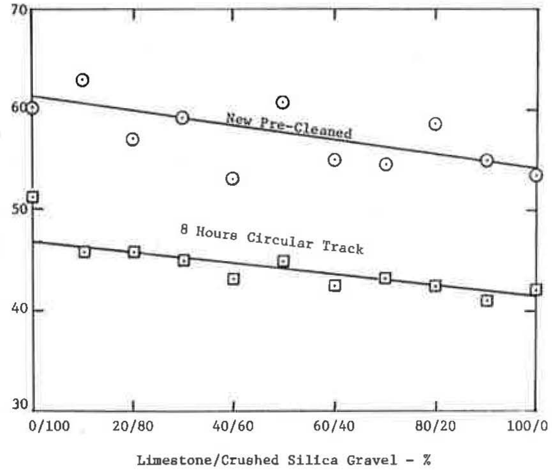
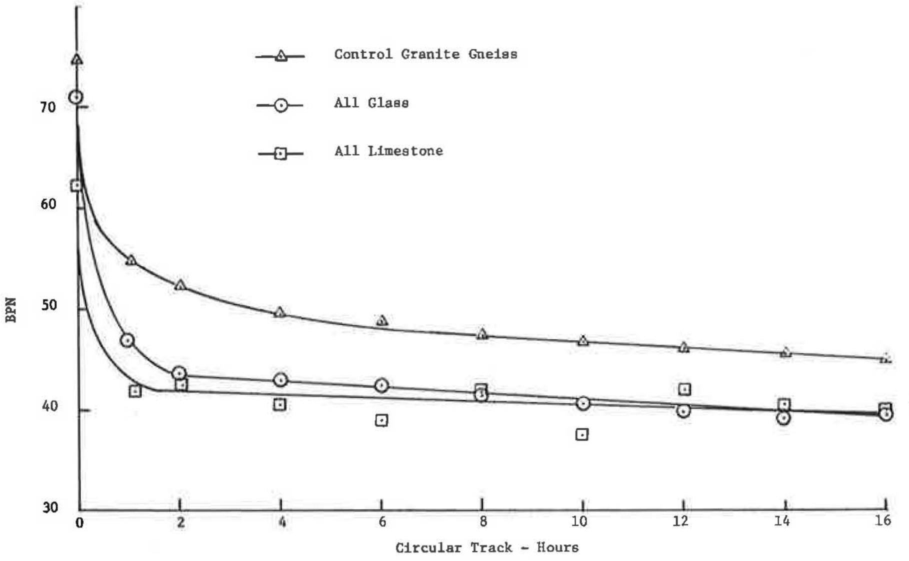


Table 3. Crushed glass, natural aggregate blend percentages.

I-2 Mix			Fine Aggregate Mix			
Glass	Limestone	Granite Gneiss[a]	Glass	Limestone	Natural Sand	Granite Gneiss[a]
100	0	0	100	0	0	0
100	0	0	50	50	0	0
50	50	0	100	0	0	0
50	50	0	0	100	0	0
0	100	0	50	50	0	0
0	0	0	50	0	50	0
100	0	0	50	0	0	50
50	0	50	100	0	0	0
0	100	0	0	100	0	0
0	0	100	0	0	0	100

[a]GN-1 laboratory control aggregate.

Figure 6. Comparison of glass, limestone, and control aggregate I-2 mixes.



were run through a laboratory jaw crusher. Larger pieces had 2 smooth molded surfaces and were flatter than what is ideal. Smaller particles resembled sharp sand grains. All sizes of the crushed glass could be handled with bare hands.

Initial trials were with all-glass mixtures that were found to polish rapidly to a relatively low BPN value. Examination of polished mixture surfaces revealed numerous flats lying parallel to the surface. The all-glass mixture results, shown in Figure 6, are compared for polishing characteristics to a polishing limestone mixture and to the laboratory control aggregate.

Blends in which 50 percent of glass coarse aggregate was replaced by another aggregate are shown in Figure 7. Some overall improvement is obtained in combining glass with the granite-gneiss control aggregate, but no benefit is gained with the coarse or fine limestone blends. What is important, however, is that the rate of polish for up to 4 hours of machine time was reduced considerably for all mixes compared to the rate of polish for the all-glass and all-limestone mixes in Figure 6. This reduction in polishing rate could prove to be significant information in light of field tests that indicated that field exposure to as many as 40 million wheel passes may not exceed wear equivalent to 2 or 3 hours of machine time (4).

In Figure 8, test results for mixes in which fine aggregate substitutions of 50 percent were made are shown together with results of 1 sand asphalt mix containing 50 percent crushed glass sand and 50 percent natural silica sand. No particular benefits were gained from blending fine aggregates in the I-2 mixes, but the sand-asphalt mix exhibited excellent early and terminal friction characteristics compared to the other mixes, including the laboratory control mix.

Experience with the glass and granite-gneiss blends indicated that blending poor polish-resistant aggregates with good polish-resistant aggregates offers a better chance to improve overall mix polishing characteristics than does blending of hard and soft aggregates both of low, terminal polish resistance.

SUMMARY

It has been possible during the course of this research to develop workable methods to determine the wear and polishing properties of aggregates and bituminous paving mixtures in the laboratory. Blending of aggregates produces an average polish resistance generally proportional to the percentage of each aggregate in the blend. Thus, it may be possible to improve marginal polish-resistant aggregates to an acceptable level by blending appropriate percentages of high polish-resistant aggregates.

ACKNOWLEDGMENTS

The work reported in this paper was sponsored by the North Carolina State Highway Commission in cooperation with the Federal Highway Administration. Research was coordinated by a liaison committee from the North Carolina State Highway Commission and the Federal Highway Administration. The work was performed by the authors in the Civil Engineering Laboratories of North Carolina State University. In addition, progress of the research has been aided by many of the personnel of the North Carolina State Highway Commission and the Federal Highway Administration as well as by several North Carolina State University faculty and staff members and students.

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.

REFERENCES

1. Mullen, W. G., Dahir, S. H. M., and Barnes, B. D. Two Laboratory Methods for Evaluating Skid-Resistance Properties of Aggregates. Highway Research Record 376, 1971, pp. 123-135.
2. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures. North Carolina State University, Highway Research Program, Proj. ERD-110-69-1, Interim Report, June 1970.

Figure 7. Comparison of I-2 mix blends with 50 percent glass coarse aggregate replacement.

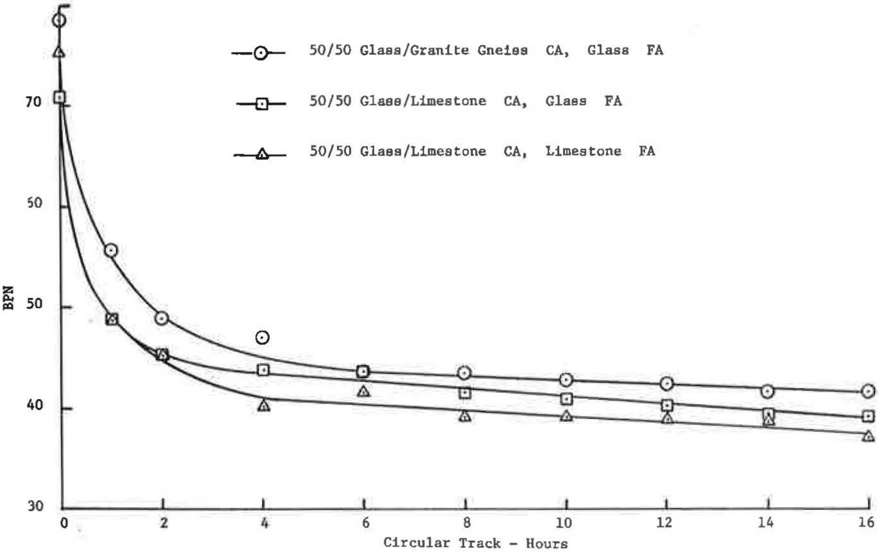
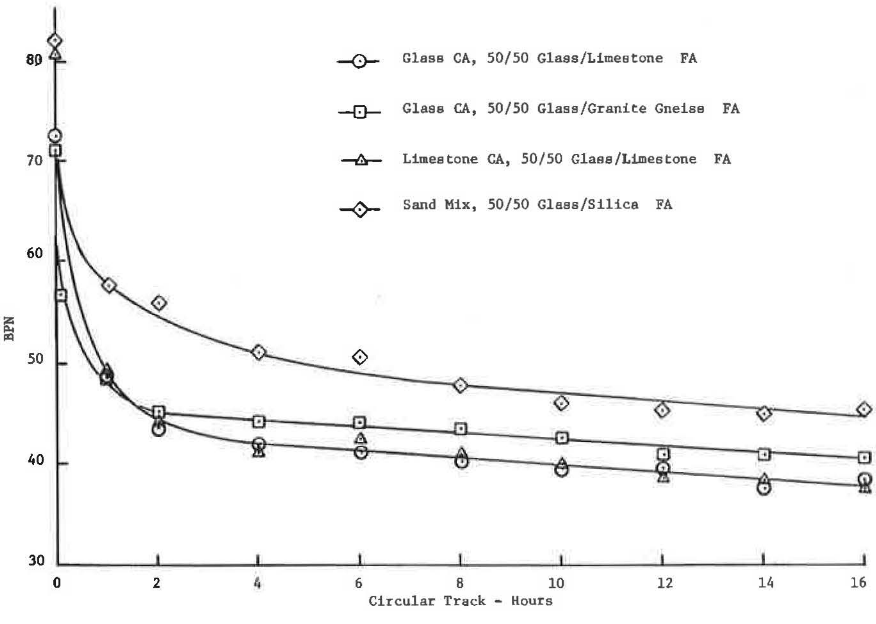


Figure 8. Comparison of I-2 mixes with 50 percent glass fine aggregate replacement.



3. Dahir, S. H. M., and Mullen, W. G. Factors Influencing Aggregate Skid-Resistance Properties. Highway Research Record 376, 1971, pp. 136-148.
4. Skid Resistance and Wear Properties of Aggregates for Paving Mixtures. North Carolina State University, Highway Research Program, Proj. ERD-110-69-1, Final Report, Sept. 1972.

PHOTO-INTERPRETATION OF PAVEMENT SKID RESISTANCE IN PRACTICE

R. Schonfeld, Ministry of Transportation and Communications, Ontario

A standard texture classification procedure has been formulated that views the pavement surface as a geometric structure expressed by 6 parameters: height, width, angularity, distribution, harshness of projections above the matrix, and harshness of the matrix itself. The function of the classification method is primarily to identify pavement surfaces and to correlate textures with the accumulated test results and experience of skid testing devices. A simple stereophotographic technique was used. Regression analyses of photo-interpreted skid numbers and skid trailer test results indicated that the coefficient of correlation ranged from 0.8 to 0.9. Examples are reported of the method's applications for examining the reason for pavement slipperiness, for obtaining skid resistance information in locations where skid test vehicles cannot operate, for investigating the skid resistance of laboratory samples of pavement textures, for easing the work load of skid trailers engaged in skid resistance surveillance, and for determining the need for correcting deficient pavement skid resistance.

•THE HIGHWAY engineer, to manipulate the surface texture in order to control the pavement's skid resistance, must have a geometric concept of the pavement surface. Pavement surface textures have been stereophotographed and studied on a macroscopic and microscopic scale by several research workers (1, 2). Real pavement surface texture includes sediments of pavement, tire, windblown debris, and other contamination. There is evidence that the real pavement texture at the microscopic end of the scale is volatile and that transitory sediments are responsible for the seasonal fluctuations of skid resistance that coincide with precipitation and change of seasons (3). The texture classification system given in Table 1 deals with surface textures as perceived by the human eye at a magnification of approximately 6. The function of the classification method is twofold:

1. To classify pavement textures by identifying the 6 geometric features of the pavement surface that are known to influence skid resistance by using the simplest means possible and
2. To use texture classification for correlating the pavement surface textures with the accumulated test results of skid testing devices.

EQUIPMENT FOR PAVEMENT SURFACE TEXTURE ANALYSIS

A 35-mm single-lens reflex camera with a focal length of 55 mm was used for taking pairs of stereophotographs. The camera was mounted on a box (Figs. 1 and 2) 457 mm above the pavement. The box was equipped with an electronic flash unit that illuminated the photographed area at an angle of approximately 45 deg. The camera was attached to a sliding seat and took pavement photographs from 2 positions 95 mm apart. The stereophotographs covered a pavement surface area approximately 10 cm square. The camera box was placed in the middle of the wheel track with the light source side of the box facing to the right of the traffic direction.

Publication of this paper sponsored by Committee on Surface Properties-Vehicle Interaction.

Table 1. Surface texture classification.

Number	Macrotexture of Projections: Approximate Height, A[a] (mm)	Approximate Width, B (mm)	Angularity, C[b]	Density of Projections as Percent of Total Area, D	Microtexture: Projection Harshness, E[b]	Background Harshness, F
0	0	16		0 to 12		Cavity in matrix surface
1	1/4	8	Round	13 to 37	Polished[c]	Polished[c]
2	1/2	4	Subangular	38 to 62	Smooth[d]	Smooth[d]
3	1	2	Angular	63 to 87	Fine grained[e]	Fine grained[e]
4	2	Less than 2 is background		88 to 100	Fine grained[f,g]	Fine grained[f,g]
5	4	Less than 2 is background			Coarse grained, subangular[f,h]	Coarse grained, subangular[f,h]
6	8	Less than 2 is background			Coarse grained, angular[f,i]	Coarse grained, angular[f,i]

[a]A texture element has a height dimension only if the surrounding area below its peak is drained.
[b]To use this chart for asphalt pavements the following adjustments should be made: If the C-parameter number is 2, then the E-parameter number should be raised by 1; if the C-parameter number is 3, then the E-parameter number should be raised by 2.
[c]No texture visible.
[d]Texture visible but microprojections too small for visual estimate of height.
[e]Height of microprojections less than ¼ mm and less than ½ their width.
[f]Microprojections must protrude by not less than ½ their width.
[g]Microprojections approximately ¼ mm high.
[h]Microprojections approximately ½ mm high or higher.
[i]Microprojections approximately ½ mm high or higher.

Figure 1. Camera box.

Figure 2. Camera box showing flash.

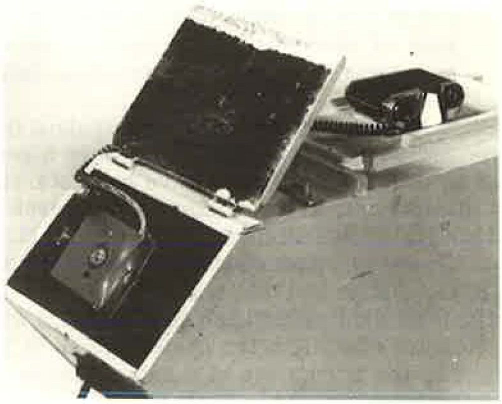
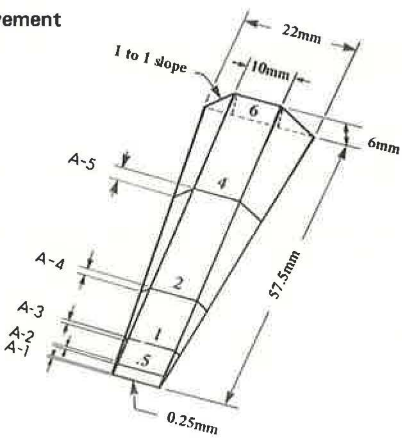


Figure 3. Vertical-scale wedge for pavement surface texture analysis.



Because pavement stereophotographs show reference scales for estimating texture dimensions, a horizontal reference scale with 1-mm divisions was attached to the light box at approximately pavement surface level. A vertical reference scale also was attached to the light box at approximately the same level. The vertical scale was wedge shaped, $\frac{1}{4}$ mm high at 1 end and 6 mm high at the other end and had 45-deg side slopes. Heights were indicated at increments of the texture height parameter (Fig. 3).

Pavement stereophotographs are viewed under a mirror-stereoscope or under a microstereoscope. If a mirror-stereoscope is used, the 35-mm pavement photographs are first enlarged into natural-scale prints. These are viewed at a magnification of 6. A microstereoscope with a magnification of 25, mounted on a light table, may be used for viewing 35-mm color transparencies (Fig. 4).

A more detailed account of pavement surface texture analysis is available (4).

TEXTURE PARAMETERS

The surface texture of a pavement was defined in terms of 6 texture parameters (Table 1).

1. Parameter A denotes the height above the matrix of projections on the pavement surface.
2. Parameter B denotes the width of the surface projections at the top of the matrix.
3. Parameter C denotes the shape of the projection, for example, round, subangular, or angular (Fig. 5). Round projections have curved sides and no edges or nearly plane sides and well-rounded corners and rounded edges. Subangular projections have somewhat blunted edges. Angular projections have sharp edges and pointed corners.
4. Parameter D denotes the density of distribution of the projections as the proportion of the whole surface area occupied by projections.
5. Parameter E denotes the harshness (microtexture) of the projections' surfaces in terms of apparent height and angularity of the microprojections, for example, polished; smooth; fine grained; coarse grained, subangular; and coarse grained, angular. A polished surface has no visible texture. A smooth surface has a visible texture, but the microprojections are too small for a visual estimate of height. A fine-grained surface has microprojections that are approximately $\frac{1}{4}$ mm high. Microprojections must protrude by not less than half their width. A coarse-grained, subangular surface has blunt microprojections that are approximately $\frac{1}{2}$ mm high or higher. Microprojections must protrude by not less than half their width. A coarse-grained, angular surface has sharp microprojections that are approximately $\frac{1}{2}$ mm high or higher. Microprojections must protrude by not less than half their width.
6. Parameter F denotes the harshness of the texture of the background surface between projections in terms of apparent height and angularity of microprojections, for example, smooth; fine grained; coarse grained, subangular; and coarse grained, angular. A cavity is an undrained area below the surface of the matrix.

The surface texture of a pavement is classified by the texture code number, which is the complete set of the 6 texture parameter numbers, for example, 2.7-1.4-1.8-55-2.1-3.0. For heterogeneous surface textures, when part of the photographed surface has been expressed in terms of 1 microtexture parameter and the other part by the full set of 6 parameters, 2 texture code numbers and their proportion are stated: 30 percent × 0-0-0-100-3.4-0 + 70 percent × 2.7-1.4-1.8-55-2.1-3.0.

ANALYSIS

A transparent grid, equivalent to 1 cm square, was placed over 1 stereophotographic print or under 1 color transparency, depending on whether a mirror-stereoscope or a microstereoscope was used. On the grid, 10 random centimeter squares were marked and numbered (Fig. 6). Each of the numbered centimeter squares of pavement surface was examined under the stereoscope, and the number of each of the 6 texture parameters was assessed according to Table 1. The numerical average of the parameter numbers of the centimeter squares was the parameter number of the photographed surface area.

Figure 4. Microstereoscope.

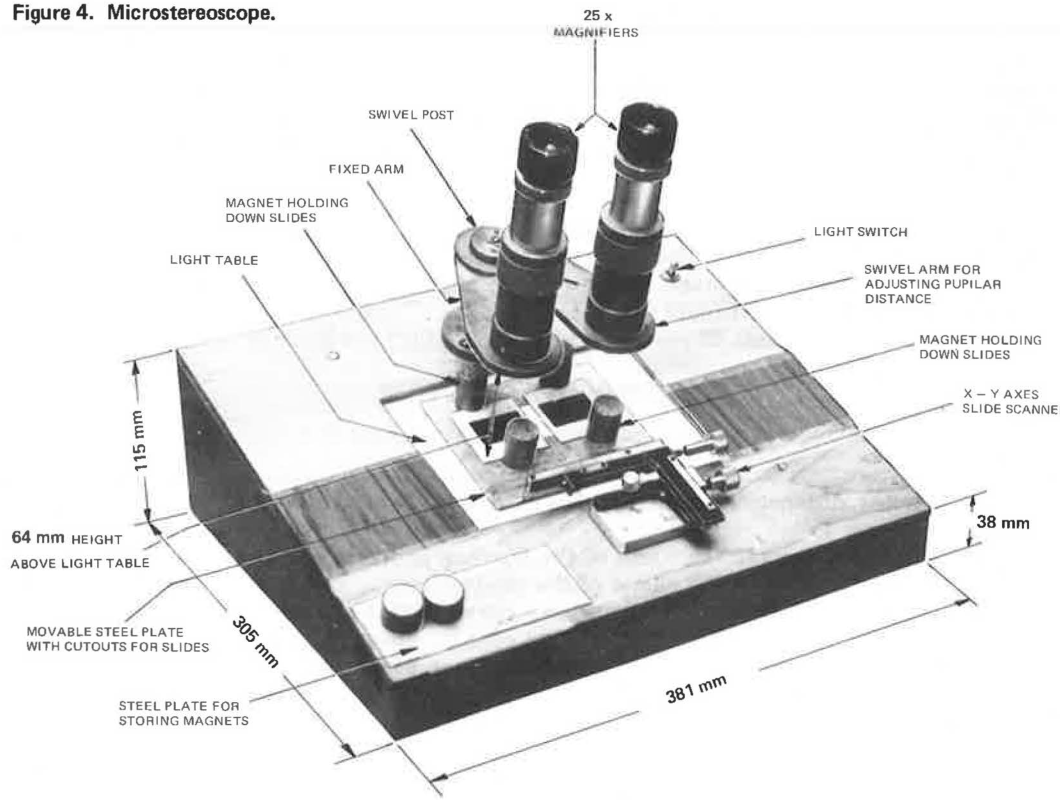


Figure 5. Angularity of projections.

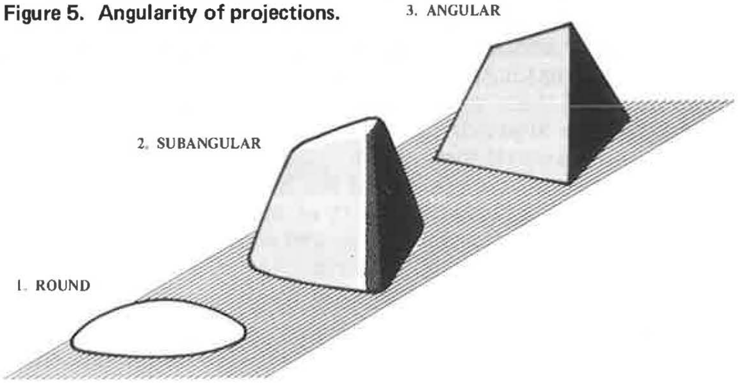


Figure 6. Transparent grid.

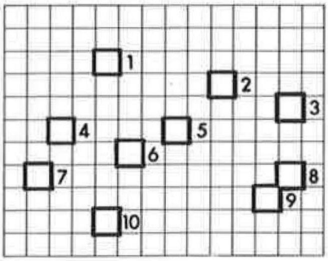
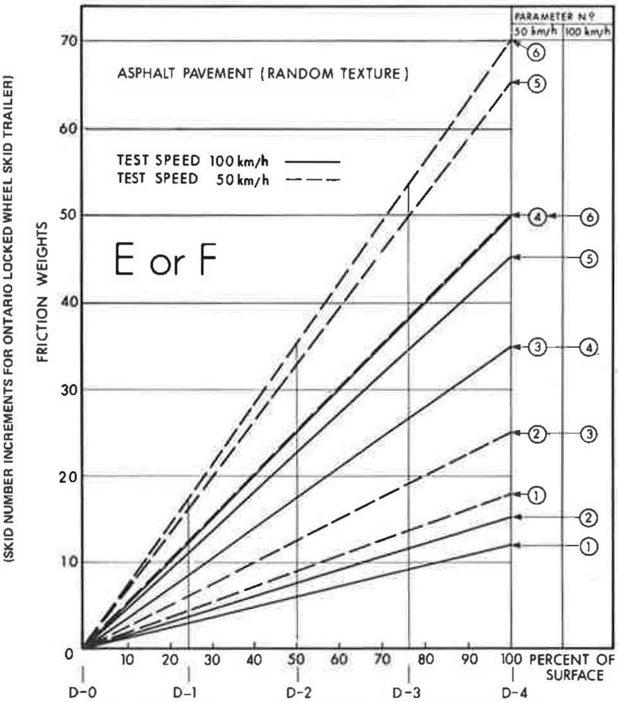


Figure 7. Friction weights of pavement texture parameter E or, if macroprojections are absent (D0), parameter F.



Note: If the density parameter is D-1, D-2, or D-3, add to the friction weight for parameter E (obtained from this graph) the friction weights for parameters A, B, and F (obtained from Figures 8, 9, and 10 respectively).

In the case of parameter C only, the average parameter number was rounded to the nearest whole number. The set of the 6 average parameter numbers was the texture code number of the photographed surface area.

SAMPLING PAVEMENT TEXTURE IN TEST SECTIONS

At least 3 pavement stereophotographs were taken in each test section. A test section is a section of pavement of uniform age and general composition that has been subjected to essentially identical wear throughout. Sharp curves and steep grades should not be included in the same test section with level tangents, nor should passing lanes be included with driving lanes.

PHOTO-INTERPRETING SKID RESISTANCE AND COMPUTING SKID NUMBERS

Correlation charts of texture code numbers and skid test results on asphalt pavements of an ASTM 2-wheel trailer were prepared (Figs. 7, 8, 9, and 10). Similar charts based on regression analyses of texture code numbers and test results of other skid testing devices may be prepared. The photo-interpreted skid number of a pavement surface is the sum of the texture parameters' friction weights obtained from photo-interpretation charts. The photo-interpreted skid numbers can either be abstracted from the charts in the ordinary way or calculated by computer. In the latter case, the computer input should consist of (a) the identification number of the pavement photograph; (b) the type of pavement; (c) the texture parameters; and (d) skid trailer test skid numbers, if available, at test speeds of 50 km/h and 100 km/h. The program should contain an option for performing a linear regression of the test skid numbers (independent variable) and photo-interpreted skid numbers (dependent variable). The output should consist of 2 listings: (a) the photo-interpreted skid numbers together with the input parameters for each photograph and the test skid number and (b) the regression analysis, namely, constant, a, and regression coefficient, b, in the equation $y = a + bx$; the coefficient of correlation; and the standard error of estimate.

EXAMINING CAUSE OF PAVEMENT SLIPPERINESS

The photo-interpretation method can throw light on the reasons for the existence of a slippery pavement condition. In the course of analyzing the pavement surface, some of the following defects became apparent:

1. Polished aggregate (texture parameter E1),
2. Excess binder or fines in matrix (texture parameter F1 or F2),
3. Excessive wear of pavement aggregate (angularity parameter C1), and
4. Stripping of aggregate (distribution parameter D1 or D2).

Example 1

The test site was located on Highway 9, with the town of Arthur to the west. The texture code number was 1.7-1.3-1.7-25 percent-2-2.7. The skid number at 50 km/h was 41; at 100 km/n, it was 26. The stone projections on the surface were low (0 to 0.5 mm), subangular to round, sparsely distributed, and smooth (partly asphalt coated). The matrix was fine grained to smooth. Therefore, skid resistance at low speed was adequate but relatively low at high vehicle speeds. Scattered flushing in the wheel paths was in evidence. Traffic will wear off the stones' asphalt coating, which will raise skid resistance. However, present flushing in the wheel tracks will probably spread and will further aggravate the friction deficiency at high speeds.

Example 2

The test site was located on Highway 6 with Rickman's Corners approximately 8 miles to the south. The texture code number was 1.5-1-1-25 percent-3.3-3. The skid number at 50 km/h was 45; at 100 km/h, it was 28. A large proportion of stone projections on the surface appeared to have broken off at, or even below, the matrix level

Figure 8. Friction weights of pavement texture parameters A, B, and F for D1.

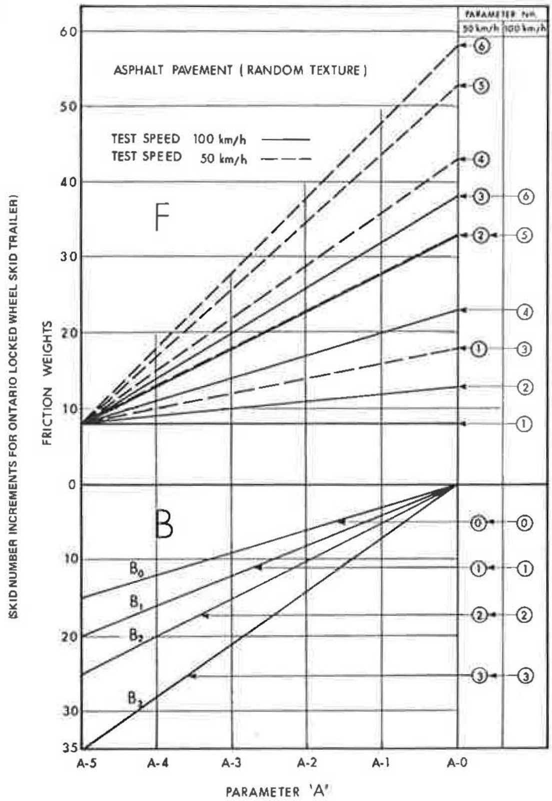


Figure 9. Friction weights of pavement texture parameters A, B, and F for D2.

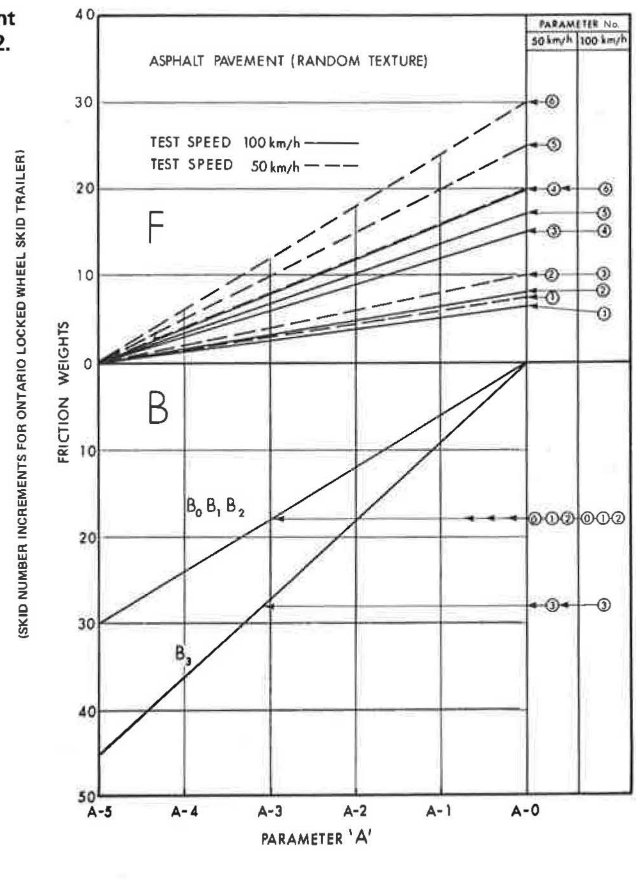


Figure 10. Friction weights of pavement texture parameters A, B, and F for D3.

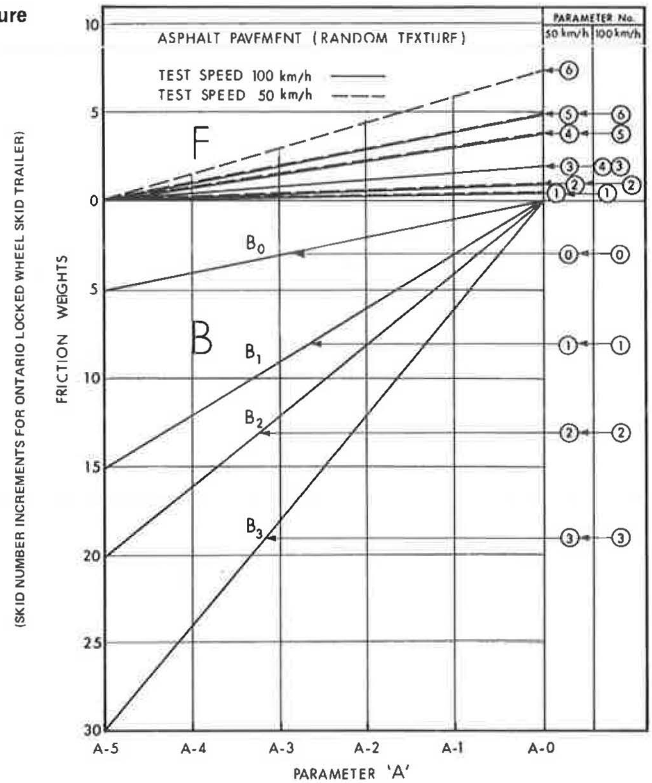


Table 2. Texture and skid-resistance changes caused by PCA wear machine, 1973.

Section Number	Pavement Wear	Parameter A		Parameter E				Parameter F				Total Change in Skid Resistance Caused by Wear Test		Skid Nu[mber] After P[CA] Wear
						Skid Resistance Change				Skid Resistance Change				
		Before PCA Wear	After PCA Wear	Before PCA Wear	After PCA Wear	50 km/h	100 km/h	Before PCA Wear	After PCA Wear	50 km/h	100 km/h	50 km/h	100 km/h	50 km/h
1	Normal sand, normal limestone	2	2	3	2	-8	-5	0/3	0/3	Nil	Nil	-8	-5	24
2	Normal sand, normal limestone	1	1	3	2	-8	-5	0/3	0/4	+2	+3	-6	-2	23
3	100 percent silica, normal limestone	2	2	3	2	-8	-5	3	3	0	0	-11	-5	28
4	67 percent silica, 33 percent normal limestone	0	0	3	3	Nil	Nil	3	2/3	-5	-3	-5	-3	40
5	33 percent silica, 67 percent normal limestone	2	2	3	2	-8	-5	3	0/3	-5	-3	-13	-8	25
6	100 percent silica, slag	3/2	0	3	3	Nil	Nil	3.5	0/7	+3	+5	+3	+5	42
7	67 percent silica, 33 percent sand, slag	0	0	3	3	Nil	Nil	3	0/4/7	+7	+7	+7	+7	41
8	33 percent silica, 67 percent sand, slag	3	3	3	3	Nil	Nil	0/3	0/3	Nil	Nil	Nil	Nil	45
9	100 percent silica, traprock	3	3	3	3	Nil	Nil	3	3	Nil	Nil	Nil	Nil	46
10	67 percent silica, 33 percent sand, traprock	3	3	3	3	Nil	Nil	3	3	Nil	Nil	Nil	Nil	52
11	33 percent silica, 67 percent sand, traprock	3	3	3	3	Nil	Nil	3	3	Nil	Nil	Nil	Nil	46
12	Normal sand, normal limestone	1	1	3	2	-8	-5	3	3	Nil	Nil	-8	-5	28

so that the height of stone projections was generally low (0 to 0.5 mm). The damaged stones were light colored and probably soft. Their presence in an HL1 (traprock) pavement was not expected. The surface of the matrix was gritty. The skid resistance of the examined section of Highway 6 met the minimum requirements at low traveling speeds but fell short of the high-speed friction needs of a curved alignment with numerous driveways. The relatively low skid resistance may be attributed to the fact that the coarse aggregate in the pavement mix had a significant proportion of white stone particles sheared off at the base. Only the traprock particles provided the desirable texture relief. The mineral composition of the coarse aggregate in this section of highway should be investigated.

EVALUATING SKID RESISTANCE OF LABORATORY AND FIELD SAMPLES

Core samples of 12 experimental concrete pavement sections containing different aggregates and mix proportions were subjected to wear under the PCA wear machine. The changes in the surface texture and consequent changes in the skid resistance of the concrete surfaces brought about by the lab test are given in Table 2. The surface changes consisted of the following:

1. Downgrading of parameter E3 (fine grained) of the coarse aggregate to E2 (smooth) in test sections 1, 2, 3, 4, and 12, in which the coarse aggregate was local limestone;
2. Downgrading of matrix parameter F3 to F1.5 and F2.5 in test sections 4 and 5 respectively; and
3. Improved skid resistance in slag sections 6 and 7, in which the slag appears to have been worn down enabling the silica-sand matrix to make contact with the tire.

ESTIMATING SKID RESISTANCE IN A CURVED ALIGNMENT

The skid trailer in a curved test path cannot be expected to realistically reflect the skid resistance of the pavement because of the lateral (centrifugal) force exerted on the test wheel's torque sensor.

The third test site was located in the eastbound collector lane on the exit ramp to Warden Avenue on Highway 401. The texture code number was 0-0-0-100-2.0-0. The skid number at 40 km/h was 27; at 50 km/h, it was 25; and at 100 km/h, it was 15. The posted speed limit was 25 mph (40 km/h). The skid resistance at this speed was extrapolated from the speed skid resistance gradient obtained from the photo-interpretation chart. The coarse aggregate was highly polished and the texture relief was less than ¼ mm. The matrix was mainly smooth; only a small proportion of it was fine grained. Therefore, skid resistance was extremely low for the friction requirement on a curved ramp.

EASING WORK LOAD OF SKID TRAILER

When only 1 skid trailer is used for extensive highway mileages (as was the case in Ontario), promptly satisfying all requests to examine highway sections separated by considerable distances is often not possible. Use of the photo-interpretation method eases the pressure exerted on the skid trailer by allowing quick diagnoses to be made from pavement stereophotographs. It can be determined whether the texture of a pavement surface

1. Excludes the possibility of a slippery condition;
2. Indicates the certainty that a slippery condition exists; or
3. Leaves doubt about the friction level, indicating that skid testing should be conducted.

DETERMINING NEED FOR CORRECTING DEFICIENT PAVEMENT

Corrective action is often undertaken when skid resistance is below the level representing the minimum frictional requirement. The list of guidelines currently used by state highway departments and the British illustrates the variety of norms (5, 6).

Determining the pavement friction coefficients required for driving tasks was undertaken in NCHRP Rept. 154 (7) with the objective of providing highway agencies with methods to determine minimum skid resistance, allowing for driver behavior. A tape switch system was developed for roadside instrumentation to measure longitudinal and lateral accelerations of vehicle maneuvers. However, it is uncertain when reliable friction requirements for traffic locations will be obtainable and whether tailor-made skid resistances will be feasible. A statistical evaluation of skid tests demonstrated that a great deal of skill and sophistication is needed to extract from survey test data results suitable for use with minimum skid resistance standards (8).

Identifying and classifying pavement surface textures and their suitability for specified traffic conditions in different climates holds the prospect of raising the skid resistance standard of our highways by eliminating those textures that have proved to have low resistance to skidding and hydroplaning. Pavement texture selection will be based on those combinations of texture parameters that must be incorporated in the surface to achieve skid resistance in the intermediate or upper scale. Therefore, determining the skid resistance standard of a pavement should be based not exclusively on a skid test result but on the accumulated testing of a particular pavement surface texture. Pavement texture can be divided into 6 groups with interpreted skid numbers in the lower, intermediate, and upper range of the known skid resistance spectrum, as follows:

1. $SN_{100\ km/h} < 20$,
2. $20 \leq SN_{100\ km/h} < 30$,
3. $30 \leq SN_{100\ km/h} < 40$,
4. $40 \leq SN_{100\ km/h} < 50$,
5. $50 \leq SN_{100\ km/h} < 60$, and
6. $60 \leq SN_{100\ km/h}$.

The 6 skid resistance ranges would be in line with the categories of friction needs which, hopefully, will be identified for different traffic situations (7). A pavement surface will be considered in need of correction if its texture has an interpreted skid resistance potential below the range of the location's friction needs.

CONCLUSION

The photo-interpretation skid resistance method has been in use in Ontario for the past 2 years in investigating highway pavements on which a skidding problem was suspected. By focusing on pavement surface geometry and by combining it with skid testing experience, the engineer is in a better position to understand the cause, prevention, and remedy of particular skidding problems.

ACKNOWLEDGMENT

Allan Ma, of the Ministry of Transportation and Communications, contributed the statistical analysis of correlated photo-interpreted skid numbers and skid trailer test results. Graham Musgrove, of the Ministry of Transportation and Communications, supplied field data and valuable advice from the beginning of the work on pavement texture classification.

REFERENCES

1. Sabey, B. E., and Lupton, G. N. Measurement of Road Surface Texture Using Photogrammetry. Transport and Road Research Laboratory, Berkshire, England, Rept. LR57, 1967.
2. Holmes, T., Lees, G., and William, A. R. A Combined Approach to the Optimization of Type and Pavement Interaction. Presented at American Chemical Society Meeting, Miami, Florida, 1971.
3. Furbush, M. A., and Styers, K. E. The Relationship of Skid Resistance to Petrography of Aggregates. Pennsylvania Department of Transportation, Bureau of Materials, Testing and Research, 1972.
4. Schonfeld, R. Pavement Surface Texture Classification and Skid Resistance. Presented at Symposium on Physics of Tire Traction, Research Laboratories of General Motors, Warren, Michigan, Oct. 1973.

5. Kummer, H. W., and Meyer, W. E. Tentative Skid-Resistance Requirements for Main Rural Highways. NCHRP Rept. 37, 1967.
6. Hawken, N., and Kennard, A. H. New Development in the Skid Resistance of Road Surface. British Ministry of Transport, London, England, 1970.
7. Determining Pavement Skid Resistance Requirements at Intersections and Braking Sites. NCHRP Rept. 154, 1974.
8. Gillespie, T. D., Meyer, W. E., and Hegmon, R. R. Skid Resistance Testing From a Statistical Viewpoint. Pennsylvania Transportation and Traffic Center, 1973.

RELATIONSHIP BETWEEN TIRE INFLATION PRESSURE AND MEAN TIRE CONTACT PRESSURE

D. J. van Vuuren, National Institute for Road Research, Pretoria, South Africa

Contact pressure is important to the design engineer because it determines the intensity of loading. However, it is not as easily monitored as tire inflation pressure. Therefore, a study was made of the relationship between tire inflation pressure and tire contact pressure. Various commercial truck tires were tested in the laboratory under various combinations of load and inflation pressures; the corresponding average contact pressures were determined by the "dirty print" approach. It was found that (a) under normal conditions of wheel load and inflation pressure the average contact pressure between the tire and the road will be less than the tire inflation pressure; (b) tire contact pressure is a function of both inflation pressure and wheel load and is a constant depending on the type of tire; and (c) the relationship between tire inflation pressure and contact pressure lies within a narrow band for the tires tested; for the combination of wheel load and tire pressures recommended by the manufacturers, an average relationship of contact pressure in kilopascals = 0.61 inflation pressure + 145.

PRESSURE on the contact between the vehicle tire and road pavement is important to the design engineer because it determines intensity of loading. For every vehicle tire, there is a fixed relationship among the internal inflation pressure, the contact pressure on the road, and the mass carried by the tire. An increase in tire contact pressure is associated with an increase in stresses in the upper layers of the pavement, which, in turn, results in greater fatigue and deformation. Pavement design engineers are interested in contact pressure, not inflation pressure, but only the latter can be monitored easily. Thus, a study was conducted to find the relationship between internal (inflation) pressure and contact pressure. Paterson (8) showed that a variation in contact pressure across the tire exists, but only the average contact pressure is discussed in this paper. All work was done statically. Lister and Nunn (7) showed that, at speeds of 8 km/h and higher, contact pressure was reduced by 2 to 5 percent. However, work done by Green, McRae, and Murphy (3) indicated that this effect is very much a function of the type of tire and that the contact pressure of the rolling wheel can be more or less than that of the static case.

THEORETICAL CONSIDERATIONS

Many pavement design engineers such as Ahlvin (1) assumed the contact pressure between the tire and the pavement to be equal to the internal pressure, that is to say, the relationship between the area of contact, A, the wheel load, W, and internal pressure, p, is

$$A = \frac{W}{p}$$

Freeme (2) concluded that this relationship is not valid and that area of contact is given more accurately as

Publication of this paper sponsored by Committee on Surface Properties-Vehicle Interaction.

$$A = \frac{W}{p+S} + A_o$$

where

A_o = additional area arising because the road is not infinitely stiff but deflects under W, and
S = stiffness factor of the tire walls.

Because a certain percentage of the load is carried to the pavement by the stiffness of the tire walls, the contact area will be smaller than that calculated by $A = \frac{W}{p}$.

The principle of the stiffness factor has been used previously; for example, Yoder (11, p. 343) assumed that, because of tire stiffness, contact pressure would be 10 percent more than inflation pressure. Other researchers found the stiffness factor not to have a constant value and that contact pressure can sometimes be more and sometimes be less than inflation pressure. Lawton (5), in a study of aircraft tires, said, "At rated tire load and an inflation pressure of 7½ psi per ply, the average contact pressure and the inflation pressure are equal. For inflation pressures higher than 7½ psi per ply, the contact pressure is less than the inflation pressure. For inflation pressures less than 7½ psi per ply, the reverse is true." Ladd and Ulery (4) also pointed to a transition but they found it to be approximately at 1000 kPa. A transition point exists, but at what pressure it will occur depends on the type of tire.

Initial tests to determine the relationship between contact pressure and inflation pressure for various truck tires led to confusing results. It was soon apparent that the 10 percent assumption was completely unacceptable. The contact pressure was often as much as 30 percent lower than the inflation pressure instead of 10 percent higher. Depending on the type of tire, tire load, and inflation pressure, contact pressure may vary from much less than the inflation pressure to much more. Other researchers such as Lister and Nunn (7) and Ledbetter, Ulery, and Ahlvin (6) have studied contact pressures and have had similar experiences.

When explaining the 10 percent assumption of Yoder (11), it is customary to indicate that a certain portion of the total load is carried through the stiff tire walls to the supporting pavement. The actual contact area is smaller than that calculated by dividing the load by the inflation pressure; the contact pressure is more than the inflation pressure. A balloon normally is used as an example of an object with no wall stiffness in which the contact pressure equals exactly the inflation pressure.

Contact Pressure of a Balloon With No Stiffness

Physical determination of inflation pressure and contact pressure of a balloon indicated that the contact pressure was always lower than the inflation pressure, sometimes by as much as 50 percent. Typical results are shown in Figure 1. The forces acting on the contact between the balloon and the pavement indicated that the contact pressure should be lower than the inflation pressure.

Figure 2 shows a balloon in contact with a smooth surface and the forces acting on it: internal pressure, external contact pressure, and tension in the balloon. Equilibrium of forces requires that

$$pA = qA + Tl \sin \theta \qquad (1)$$

where

q = average contact pressure,
T = tension per unit length (at the edge of the contact),
θ = angle between T and the horizontal,
l = circumference of A, and
D = contact diameter.

Figure 1. Relationship between inflation pressure and contact pressure for a balloon.

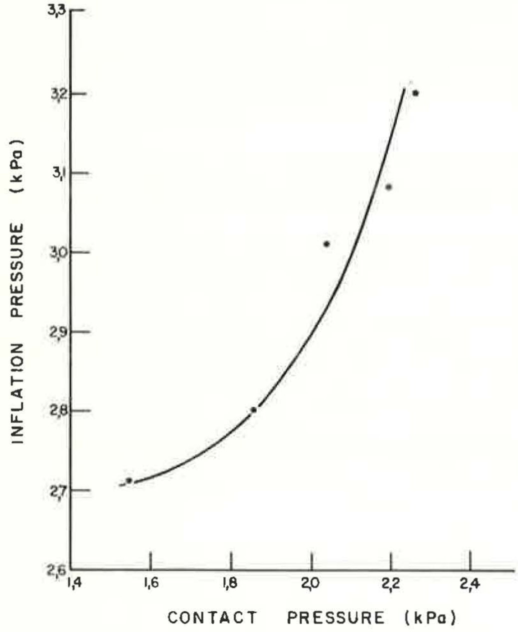
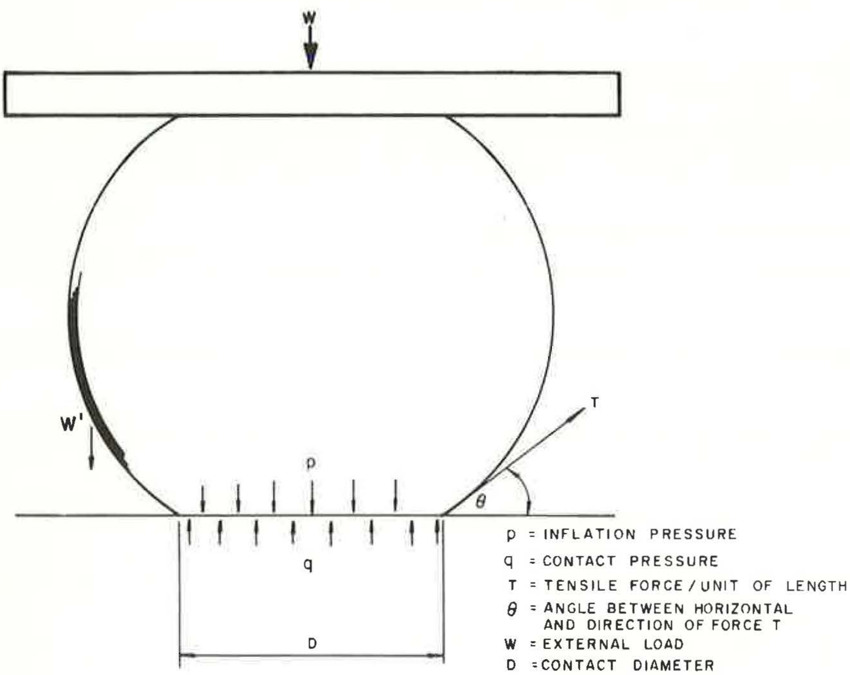


Figure 2. Forces acting on a balloon in contact with a smooth horizontal plane.



Assuming a circular area of contact

$$l = \pi D$$

and

$$A = \frac{\pi}{4} D^2$$

Eq. 1 reduces to

$$q = p - \frac{4T}{D} \sin \theta \qquad (2)$$

Therefore, q must be smaller than p as long as T, D, and θ are positive.

Contact Pressure of a Ball With Some Wall Stiffness

When a stiffer balloon is loaded, a certain portion of the load is carried to the pavement by the stiff walls (W′ in Fig. 2). This stiffness contribution can be determined by loading the ball at zero internal pressure and measuring the load to deform the ball to the same contact area as when it is inflated. If this load is W′, Eq. 2 becomes

$$q = p - \frac{4T}{D} \sin \theta + \frac{W'}{A} \qquad (3)$$

As T is a function of p and W, q is a function of both p and W. That is,

$$q = f_1(p) + f_2(W, p) \qquad (4)$$

where

$f_1(p)$ = function of inflation pressure, and
$f_2(W, p)$ = combined function of inflation pressure and load.

Average Contact Pressure of a Vehicle Tire

The relationship between inflation pressure and average contact pressure is considerably more complex when a vehicle tire is analyzed (Fig. 3) because the problem is bisymmetrical and, as a result of the physical thickness of the rubber, there is p distribution from inside to outside making a, the imaginary internal contact area, and A very different. The exact distribution also is not known and the relationship cannot be calculated. Figure 3 shows the forces acting on the tire and the contact between the tire and the ground.

Equilibrium of forces requires that, for the tire,

$$Q = qA \qquad (5)$$

and, at the contact between tire and smooth pavement,

$$qf_1(B, L) + 2T_1 \sin \theta_1 f_2(\ell) + 2T_2 \sin \theta_2 f_3(b) = pf_4(b, \ell) + C \qquad (6)$$

where

$f_1(B, L)$ = A, which is a function of B and L;
B = contact width;
L = contact length;
T_1 = tensile stress per unit length in the tire wall;
θ_1 = angle between the horizontal and the force direction of T_1;
$f_2(\ell)$ = projected length over which T is active;

Figure 3. Forces acting on a vehicle tire in contact with a smooth horizontal plane.

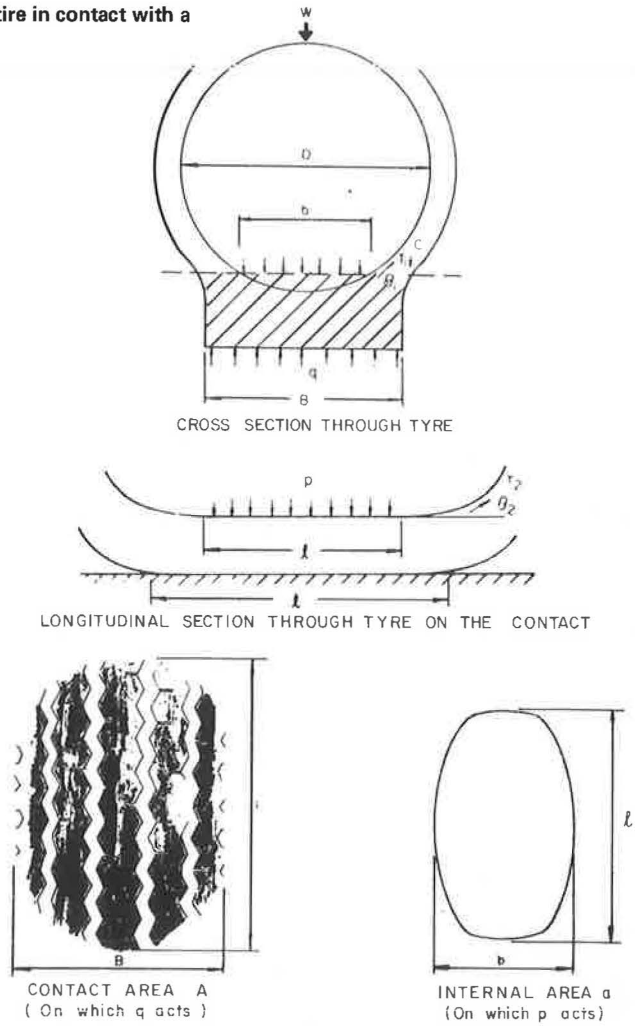
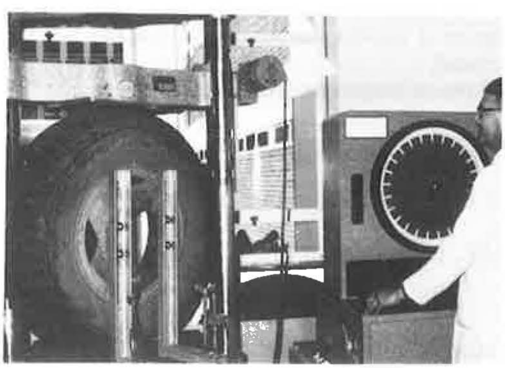


Figure 4. Test tire in Baldwin press.



ℓ = imaginary length inside the tire over which p acts;
T_2 = tensile stress per unit length in the tire base;
θ_2 = angle between the horizontal and the force direction of T_2;
$f_3(b)$ = projected length over which T_2 is active;
b = imaginary pressure width;
$f_4(b, \ell)$ = a, imaginary contact area inside the tire, which is a function of b and ℓ; and
C = component of load that is transmitted through the tire wall.

So, the formula is again a complex form:

$$q = f_1(p) + f_2(p, W) + C' \tag{7}$$

C′ will depend on factors such as tire dimensions, ply rating, rubber thickness and hardness, and tread pattern.

PRACTICAL MEASUREMENT

Experimental Work

The relationship between tire inflation pressure and tire contact pressure was determined empirically for 6 highway tires ranging in size from 7.50-15 × 10 to 11.00-22 × 14. This represents the bulk of tires used on heavy commercial vehicles in South Africa. The method was as follows. A truck tire was mounted in a Baldwin press (Fig. 4) and left for 24 hours to attain equilibrium at a given temperature. The tire was then loaded to the desired load and a print of the contact area was obtained on a smooth surface. The printing medium was ordinary black shoe polish. Two areas could then be measured—the apparent contact area and the actual contact area. The former is the total area included in the envelope of the contact print including points of no contact; the latter is areas of contact only (Fig. 5). Although calculations that use the actual contact area will give the actual contact pressure on the pavement surface, this is of interest only for the uppermost portion of the surface, and the stress concentrations disappear at a shallow depth.

Combinations of load and inflation pressure ranging between 9.0 and 17.0 kN and 300 and 500 kPa respectively were used, and the corresponding contact areas were determined.

Figure 6 shows typical load versus contact area curves at various inflation pressures for an 8.25-20 × 10-ply tire. (This tire will be used as an example throughout this paper.) The test was performed at 43 C. Similar curves also were obtained at 25 C. In Figure 7, these results are replotted as inflation pressure versus contact pressure curves for various loads. Taking the average of results obtained at 25 C and 43 C (the difference between these curves is about ±3 percent), we obtain a formula that gives the relationship among q, p, and W for this tire:

$$q = (0.013p + 10.5)\,W + 0.119p + 125.9 \tag{8}$$

The conclusion that contact pressure depends on wheel load, inflation pressure, and, to a large degree, the constant C′ can be drawn from this relationship. Within the range of testing, this formula is valid for any combination of wheel load and tire inflation pressure. The tire manufacturers, however, recommend a certain relationship between inflation pressure and tire load as shown in Figure 8 (9). Truck operators tend to follow these recommendations because they ensure the longest tire life. This relationship is given by the formula

$$W = 0.228p + 4.98 \tag{9}$$

Inserting Eq. 9 into Eq. 8 gives

$$q = 0.0003p^2 + 0.417p + 175.7 \tag{10}$$

Figure 5. Differentiation between apparent and actual contact areas.

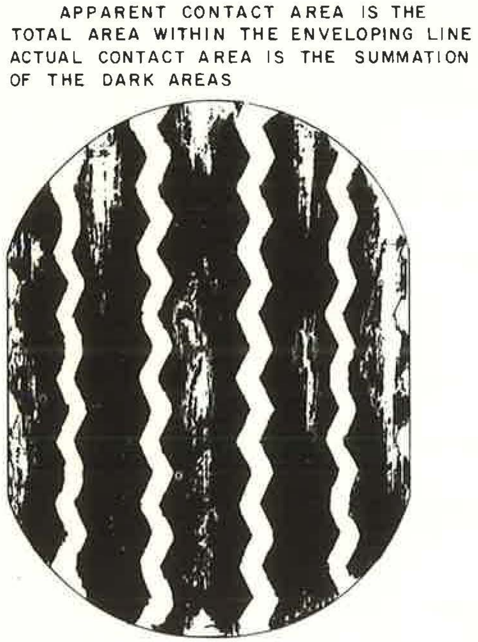


Figure 6. Relationship between contact area and load for an 8.25-20 x 10-ply tire at 43 C.

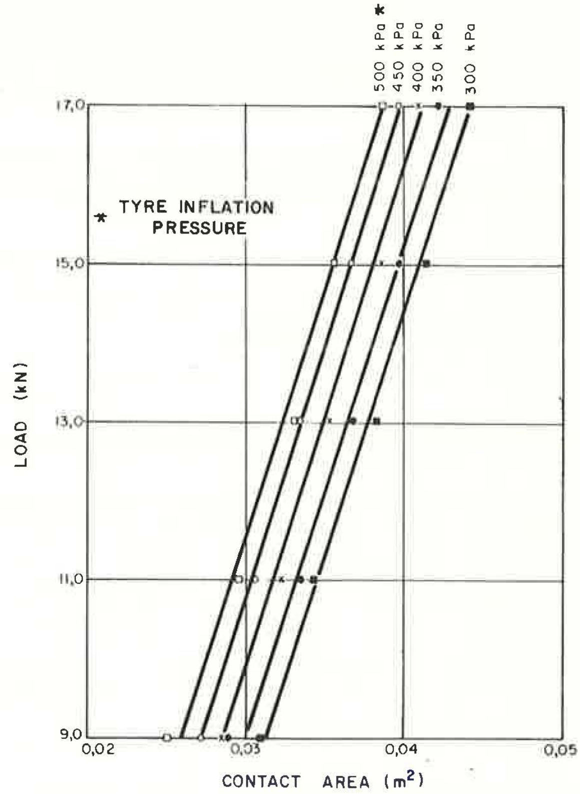


Figure 7. Relationship between tire inflation pressure and contact pressure for an 8.25-20 x10-ply tire at 43 C.

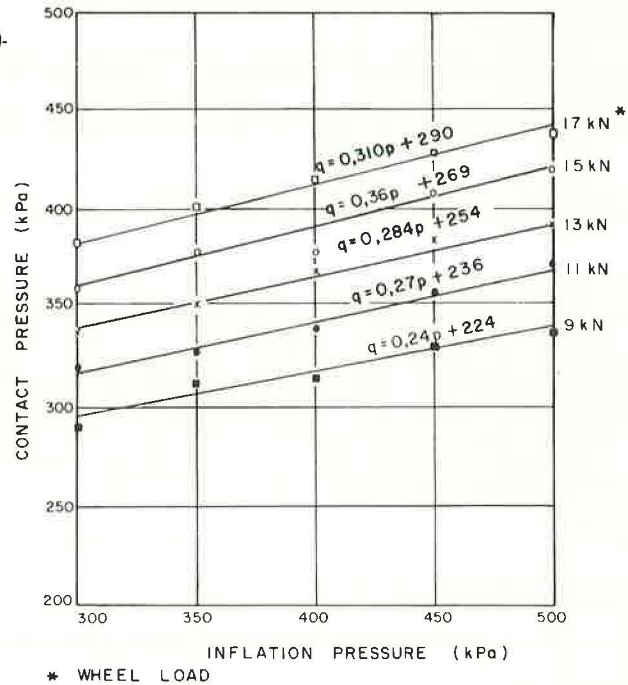


Figure 8. Relationship between recommended load and inflation pressure for an 8.25-20 x 10-ply tire.

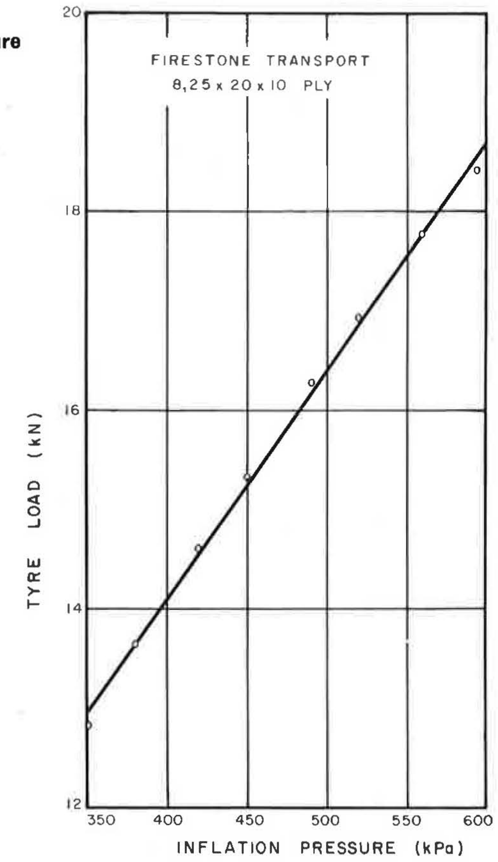
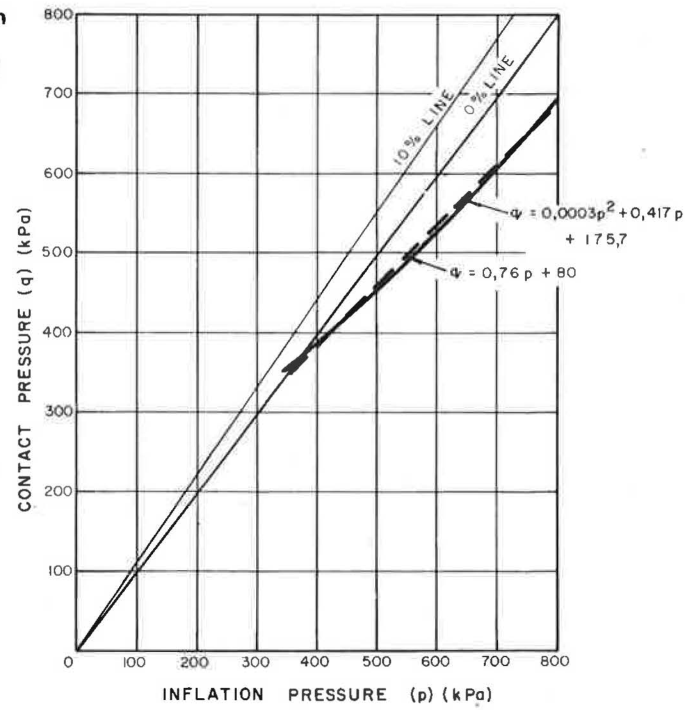


Figure 9. Relationship between inflation pressure and contact pressure for an 8.25-20 x 10-ply tire (for recommended combinations of load and inflation pressure).



This equation, which represents the relationship between contact pressure and inflation pressure, is shown in Figure 9. The 10 percent and 0 percent assumption lines are also shown. From this figure the transition point can be clearly discerned; at low inflation pressures, where the stiffness of the tire wall is predominant, the contact pressure is higher than the inflation pressure, but above about 350 kPa the contact pressure is lower because, after this point, the tire walls go completely into tension and their stiffness is no longer important.

The test range equation, Eq. 10, can be approximated by the much simpler straight-line equation

$$q = 0.76p + 80 \tag{11}$$

This is most important because the contact pressure will be a constant 76 percent of the inflation pressure for the entire practical range of wheel load and inflation pressure.

Effect of Changes in Wheel Load and Inflation Pressure on Contact Pressure

This effect can be studied by using Eq. 8. Figure 10 shows the effect on contact pressure of changes in inflation pressure for values W of 12.6 kN and 18.0 kN, which are the minimum and maximum values given by Tredco (9). It is obvious that a change in inflation pressure will result in a change in the contact pressure of approximately a third. Higher inflation pressures thus may not have the deleterious effect on pavements as was commonly believed. Operators who formerly enjoyed special axle-load exemptions because they operated at very low inflation pressure may now have to forfeit this privilege because very low inflation pressure is not associated with an equally low contact pressure.

Figure 11 shows the effect on contact pressure of changes in wheel load at constant inflation pressure. This figure, more than any other, contradicts the belief that a direct and general relationship between contact pressure and inflation pressure exists, that the contact pressure is directly proportional to the wheel load. Van Vuuren (10) has indicated that most damage to road surfaces is done by light wheel loads operating at high inflation pressures. Although this is correct, Figure 11 indicates that the effect will, in practice, be less than anticipated previously. For example, when a truck operates fully loaded in 1 direction, it will have a certain contact pressure associated with its inflation pressure. When it returns empty with a lower wheel load but the same inflation pressure (pressure inside a tire does not change significantly with a change in wheel load), it will operate with a greatly decreased contact pressure, and the resultant damage to the surfacing will be much less than anticipated.

All of this, including Eqs. 8, 10, and 11, applies only to the 8.25-20 × 10-ply tire tested. Similar equations for all the tires tested are given in Tables 1, 2, and 3. Figure 12 shows a combination of relationships between contact pressure and inflation pressure for all the tires tested for combinations of load and inflation pressure as directed by Tredco (9). It is interesting to find them all clustered together within a narrow envelope. Figure 13 shows the average line through all these curves, and I recommend that this line be used as the general relationship between tire contact pressure and tire inflation pressure for tires used on road vehicles.

CONCLUSIONS

Under normal combinations of wheel load and inflation pressure, the average contact pressure between the tire and the road will be less than the tire inflation pressure.

At constant inflation pressure, the contact pressure varies with load. A 100 percent increase in load normally is associated with a 30 to 40 percent increase in contact pressure.

At constant load, a 100 percent increase in inflation pressure will be associated with an average increase of 30 percent in contact pressure.

If both wheel load and tire inflation pressure change in accordance with the tire manufacturer's recommendations, a change in inflation pressure will be associated with a 60 percent change in the contact pressure.

Figure 10. Relationship between tire inflation pressure and contact pressure for constant values of wheel load.

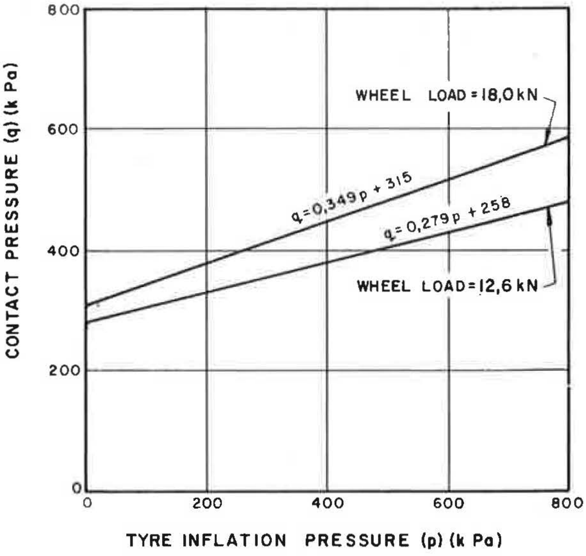


Figure 11. Relationship between contact pressure and wheel load for constant values of inflation pressure (8.25-20 x 10-ply tire).

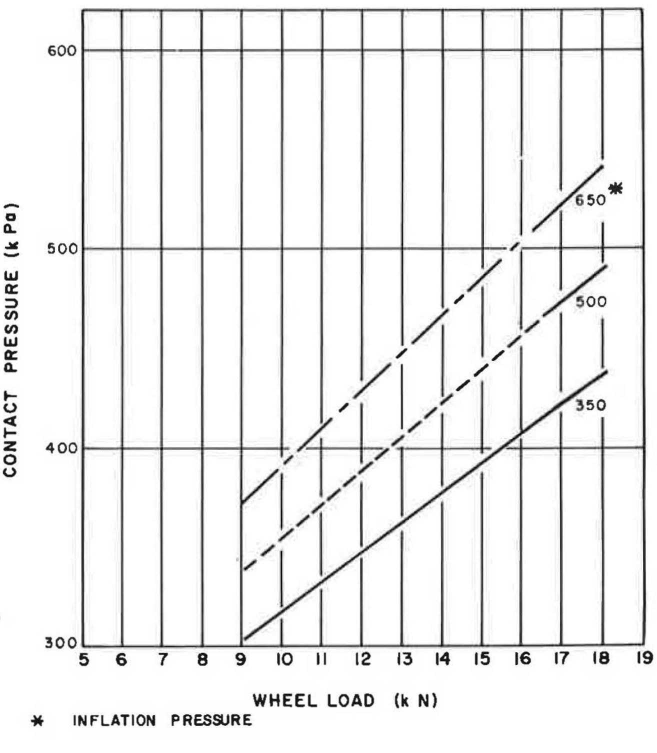


Table 1. Relationship of contact pressure, inflation pressure, and wheel load for Eq. 8.

Tire	Relationship
7.50-15 Michelin Radial	q = (-0.0029p + 13) W + 0.52p + 52
8.25-20 × 10 Firestone Transport	q = (0.013p + 10.5) W + 0.119p + 125.9
9.00-20 × 10 Firestone	q = (0.024p - 0.9) W - 0.001p + 259.6
10.00-20 × 14 Papleguas Goodyear Brazil	q = (0.002p + 6.8) W + 0.33p + 110.0
11.00-20 × 14 General SDT	q = (0.008p + 2.3) W + 0.04p + 313.0
11.00-22 × 14 General Jet Cargo	q = (0.009p + 2.6) W + 0.098p + 211.0

Table 2. Relationship between contact pressure and inflation pressure for Eq. 10.

Tire	Relationship
7.50-15 Michelin Radial	$q = 0.00005p^2 + 0.71p + 134$
8.25-20 × 10 Firestone Transport	$q = 0.0003p^2 + 0.417p + 176$
9.00-20 × 10 Firestone	$q = 0.0006p^2 + 0.109p + 256$
10.00-20 × 14 Papleguas Goodyear Brazil	$q = 0.00005p^2 + 0.530p + 161$
11.00-20 × 14 General SDT	$q = 0.0002p^2 + 0.170p + 331$
11.00-22 × 14 General Jet Cargo	$q = 0.0003p^2 + 0.258p + 219$

Table 3. Approximate linear relationship between contact pressure and inflation pressure for Eq. 11.

Tire	Relationship
7.50-15 Michelin Radial	$q = 0.66p + 145$
8.25-20 × 10 Firestone Transport	$q = 0.76p + 80$
9.00-20 × 10 Firestone	$q = 0.66p + 134$
10.00-20 × 14 Papleguas Goodyear Brazil	$q = 0.38p + 194$
11.00-20 × 14 General SDT	$q = 0.27p + 380$
11.00-22 × 14 General Jet Cargo	$q = 0.37p + 292$

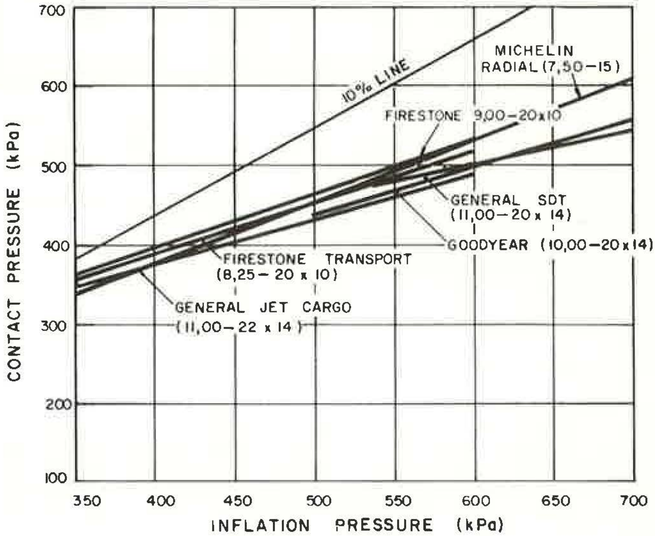


Figure 12. Relationship between contact pressure and inflation pressure for various tires (recommended combinations of wheel load and inflation pressure).

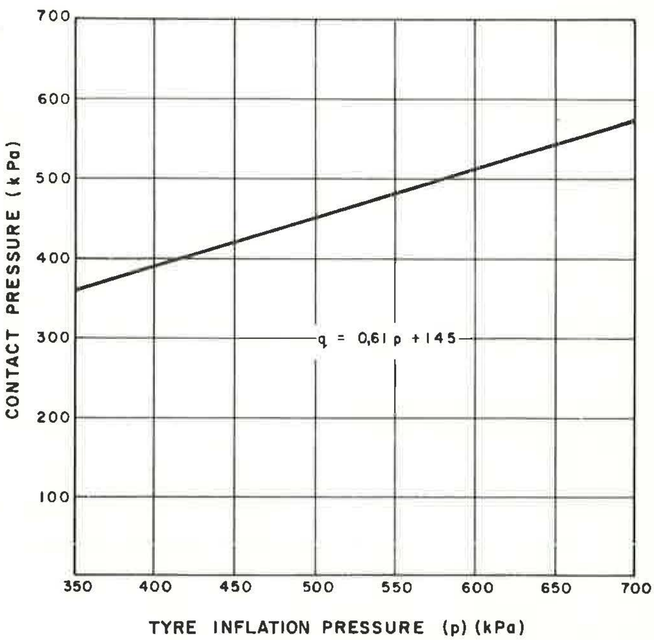


Figure 13. Average straight-line relationship between contact pressure and inflation pressure (for recommended combinations of wheel load and inflation pressure).

REFERENCES

1. Ahlvin, R. G. Developing a Set of CBR Design Curves. U.S. Army Corps of Engineers, Instruction Rept. 4, Nov. 1959.
2. Freeme, C. R. The Behaviour of Bituminous Surfacings in Asphalt Pavements. University of Natal, South Africa, PhD dissertation, 1972.
3. Green, A. J., McRae, J. K., and Murphy, N. R. Stresses Under Moving Vehicles: Distribution of Stresses on an Unyielding Surface Beneath Stationary and Towed Pneumatic Tires. U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Tech. Rept. 3-545, 1964.
4. Ladd, D., and Ulery, H. H. Aircraft Ground Flotation Investigation. U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, July 1967.
5. Lawton, W. L. Static Load Contact Pressure Patterns Under Airplane Tires. HRB Proc., Vol. 36, 1957, pp. 233-239.
6. Ledbetter, R. H., Ulery, H. H., and Ahlvin, R. G. Traffic Tests of Airfield Pavements for the Jumbo Jets. Proc., Third Internat. Conf. on Structural Design of Asphalt Pavements, London, 1972.
7. Lister, N. W., and Nunn, D. E. Contact Areas of Commercial Vehicle Tyres. Her Majesty's Stationery Office, London, Transport and Road Research Laboratory Rept. LR 172, 1968.
8. Paterson, W. D. O. Traffic Compaction of Asphalt Concrete Surface Courses. Univ. of Canterbury, New Zealand, PhD dissertation, 1972.
9. Engineering Data—Tyres and Rims. In Tredco Yearbook, Johannesburg, South Africa, 1971.
10. Van Vuuren, D. J. The Allowable Wheel and Axle Mass of Heavy Vehicles. Univ. of Pretoria, South Africa, DSc dissertation, 1972.
11. Yoder, E. J. Principles of Pavement Design. John Wiley and Sons, Inc., New York, 1959.

EVALUATION OF OPEN-GRADED PLANT-MIX SEAL SURFACES FOR CORRECTION OF SLIPPERY PAVEMENTS

Verdi Adam and S. C. Shah, Louisiana Department of Highways

This paper presents an extended evaluation of skid characteristics of various experimental plant-mix seal (PMS) and dense-graded hot-mix surfaces. The evaluation was performed on 1,000-ft (304.8-m) duplicate sections on US-190 in the Baton Rouge area. The sections consisted of a dense-graded gravel hot-mix surface of approximately 1-in. (25-mm) thickness and 3 PMS surfaces (crushed gravel, slag, and expanded clay) of $^5/_8$-in. (16-mm) thickness. Evaluation of these surfaces consisted of skid measurements according to ASTM E 274. Four-year data on the frictional performance of these surfaces have indicated that, in general, the PMS surfaces seem to possess the most desirable features with respect to constructability, skid resistance, texture, and drainage; specifically, the expanded clay and slag PMS surfaces show higher initial skid numbers and are able to maintain these numbers over an extended period of time under light and heavy traffic conditions; in addition to providing high skid resistance these PMS surfaces tend to reduce the potential for hydroplaning and ice glaze; and, in general, PMS surfaces provide flatter friction-speed gradients than does the corresponding hot-mix surface. The paper also provides information on the factors to be considered during the design and construction of these PMS surfaces and supplementary specifications for their design, construction, and acceptance.

•SKIDDING is caused by the interaction of 3 factors: the driver, the vehicle, and the roadway. Although it has not been possible to isolate human factors that contribute to skidding, considerable effort has been expended to accurately represent the roadway factor with emphasis on the frictional properties of the pavement surface and on the mechanism for surface renewal and prolonged high skid resistance. This paper is concerned with Louisiana's approach to providing skid-resistant surfaces by using bituminous surfaces.

MECHANISM FOR PROVIDING SKID RESISTANCE

A pavement becomes slippery when lubrication exists in the tire-surface contact area, when the inherently high skid resistance of a new surface has been worn or polished away by traffic, or when vehicle speeds are high enough to reduce hydrodynamically the tire-surface contact below the level required for vehicle maneuver (1). The mechanism for providing and maintaining prolonged high skid resistance should use an aggregate-binder combination that would provide adequate microtexture in the pavement and ample and quick water drainage.

In 1969, Louisiana initiated a study to determine which mechanism would provide the best nonskid surface and maintain the lowest skid decay rate. Figure 1 is a block diagram of the methods and materials that are generally advocated to improve surface skid resistance. Louisiana has had previous experience in the design and construction

Publication of this paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction.

Figure 1. Bituminous surfacing methods to correct slippery pavements.

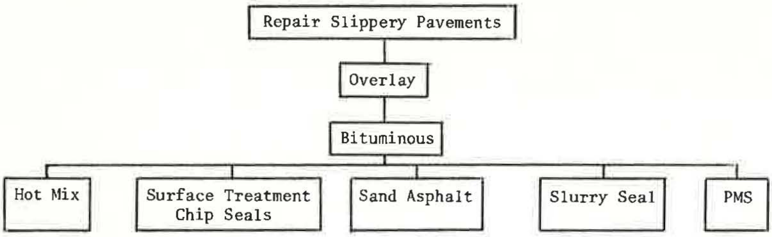


Table 1. Mix design data.

Type	Gravel Sand Hot Mix	Gravel PMS[a]	Gravel PMS[b]	Expanded Clay PMS	Slag PMS
Aggregate, percent	95	93	93	84	91
Asphalt cement, percent	5	7	7	16	9
Crushed on No. 4, percent	79	90	95	—	—
Asphalt cement grade	60-70	60-70	60-70	60-70	60-70
Laboratory specific gravity	2.34	—	—	—	—
Roadway, percentage of laboratory specific gravity	96.3	—	—	—	—
Voids, percent	5.3	—	—	—	—
Voids filled with asphalt, percent	68.2	—	—	—	—
Marshall stability, pounds	1,885	—	—	—	—
Flow, 1/100-in.	12	—	—	—	—

Note: 1 lb = 4.448 N. 1 in. = 25.4 mm.

[a]75 percent crushed faces. [b]95 percent crushed faces.

Table 2. Extracted aggregate gradation percent passing.

Sieve Size	Gravel Sand Hot Mix	Gravel PMS[a]	Gravel PMS[b]	Expanded Clay PMS	Slag PMS
$^3/_4$ in.	100	—	—	—	—
$^1/_2$ in.	98	100	100	100	100
$^3/_8$ in.	86	96	98	98	98
No. 4	59	53	46	49	41
No. 10	44	26	13	10	6
No. 16	—	—	—	—	—
No. 40	30	11	4	2	2
No. 50	—	—	—	—	—
No. 80	13	—	—	—	—
No. 100	—	2	—	1	1
No. 200	9	—	1	—	—

Note: 1 in. = 25.4 mm.

[a]75 percent crushed faces. [b]95 percent crushed faces.

Table 3. Summary of frictional performance, exterior lane, high traffic volume.

Age (months)	Cumulative Traffic (millions of vehicles)	Skid Numbers at 40 mph by Type of Surface			
		Gravel Sand Hot Mix	Gravel PMS	Expanded Clay PMS	Slag PMS
0	0	36	42	60	52
4	0.68	37	47	60	56
12	2.11	42	46	59	51
18	3.90	39	45	55	51
32	5.83	46	46	57	51
40	6.83	44	44	56	50
48	7.83	41	43	52	52

Note: 1 mile = 1.6 km.

of surfaces with all but the plant-mix seal (PMS) method the success of which was shown as early as 1967 by Mills (2) who reported on the frictional performance of these surfaces and compared PMS surfaces to dense-graded hot-mix surfaces with respect to skid resistance, drainage, and other characteristics.

DESCRIPTION OF TEST SECTIONS

Eleven different types of bituminous surfaces were used on US-190, a 4-lane divided highway in Baton Rouge. The project consisted of 10 duplicate 1,000-ft (304.8-m) sections. These were constructed on both the exterior and the interior lanes of the westbound roadway, which had an average daily traffic (ADT) of 9,510. The sections consisted of 3 hot-mix sections approximately 1 in. (25 mm) thick and 4 PMS sections ⅝ in. (16 mm) thick. Sand asphalt and slurry-seal mixes made up the rest of the sections and were approximately ¾ in. (19 mm) and ⅜ in. (10 mm) thick respectively. However, this paper will discuss only the dense-graded hot-mix and PMS sections. The hot-mix sections were made up of dense-graded mix consisting of crushed gravel and mixed with mineral filler and asphalt cement. The PMS sections consisted of 2 sections with crushed gravel, 1 with 75 percent crushed faces, and 1 with 95 percent crushed faces; the expanded clay aggregate PMS; and the slag PMS. Mix design data for these sections are given in Tables 1 and 2.

DESIGN OF MIXES

For asphalt concrete mixes, the Marshall method was used to determine asphalt content. The PMS mixes, because of their open texture and lack of mastic, did not lend themselves to the application of this design criterion and, therefore, optimum asphalt content was determined subjectively by visual observation. Briefly, the aggregate-asphalt mixtures were prepared with 3 asphalt contents, and the mix that indicated maximum aggregate coating with minimum asphalt runoff was selected as having optimum design asphalt content.

CONSTRUCTION CONTROL

At the plant, the problems associated with the control of asphalt concrete mixtures with respect to temperature and physical properties were minimal. However, PMS mixtures required special handling because of their higher asphalt contents and absence of fine material. To ensure adequate film thickness of the binder around the aggregate particles, we had to limit the mix temperature to 260 F (126.7 C). In 1 instance in which the temperature was about 300 F (148.9 C), the asphalt tended to run off the aggregate particles into the bottom of the truck. Although this did not prove detrimental to the mix on the roadway, it did cause some truck cleaning problems. These PMS mixtures, because of their open texture and absence of mineral filler, which is recognized as an antistrip additive, required a small amount of antistrip additive in the asphalt cement before mixing. Redicote 80S, 0.5 percent by weight of asphalt, was specified for these mixtures.

All sections were constructed in 1 lift. The mixture spreader had automatic screed control to the approximate thickness of 1 in. (25 mm) for hot mix and ⅝ in. (16 mm) for PMS.

The asphalt concrete mixtures were rolled by a tandem, a pneumatic, and a tandem roller, in that order. The PMS sections were rolled first with the tandem and then with the pneumatic rollers.

EVALUATION OF SURFACES

Major emphasis was placed on evaluating the skid, texture, and drainage characteristics of the various PMS surfaces.

Skid Performance

The frictional performance of the various surfaces with time and under wet pavement condition is indicated in Tables 3 and 4. The skid number (SN) relates to the friction factor f as determined by ASTM E 274. SN is the quantity 100 f; f = F/L where F is obtained in a strictly defined manner according to ASTM (3). Figures 2 and 3 show the graphical relationship of data given in these tables. The data represent 48-month skid measurements at 40 mph (64.4 km/h). The data collection on the PMS section with 75 percent crushed faces was discontinued because of unfavorable SNs. This section, therefore, is not referred to in the figures. The figures demonstrate the superior SN characteristic of the PMS surfaces constructed with manufactured aggregate. The gravel PMS, even with 95 percent crushed faces, failed to match the initial as well as the 48-month SN values of these 2 surfaces. Any corrective measure for control of surface slipperiness should have at least 2 characteristics: high initial skid resistance and no reduction of skid resistance with time and traffic. After 48 months of exposure to traffic (12 million vehicles), PMS surfaces have retained high frictional characteristics.

Essentially, traffic polishes aggregate. This reduces microtexture, which, consequently, reduces SN. If a renewal mechanism for surface microtexture is built into the pavement surface, prolonged skid resistance can be maintained. Expanded clay and slag aggregate seem to possess these surface renewal properties and are able to provide and maintain high, uniform skid resistance. However, these properties are lacking in the gravel PMS surface. The 48-month SNs for expanded clay and slag PMS, as demonstrated by both lanes (heavy and light traffic), seem to agree with the SNs reported by Mills (2). Furthermore, the average SN of 45 for asphalt concrete reported by Mills (2) also closely agrees with the present data. This comparison with Mills's data points out the magnitude of the SNs of PMS surfaces in general, but it is not based on the material component system (specifically with respect to aggregate type and asphalt content) used in the 4 western states of his study.

A surface should have a third characteristic for it to be an acceptable corrective mechanism, that is, it should show little or no decrease in skid resistance with increasing speed. Figure 4 is a speed-SN plot for hot-mix and PMS surfaces. The data represent 12-month SNs in an exterior lane. The expanded clay PMS surface does not follow the generally recognized association of flatter gradients with open texture. Possibly the expanded clay section, because of a higher than normal asphalt content, had shown some flushing and a change in texture, which seemed tighter than that observed during the initial period. The friction speed gradients for 48-month data had similar flat characteristics to those in Figure 4.

Texture and Surface Drainage

Most of the skid characteristics of the PMS surfaces are derived from microtexture, macrotexture, and surface drainage. Texture is a necessary prerequisite for generating friction, but it also provides channels by which water can escape from under the tire. Microtexture is what makes the aggregate particle feel gritty or smooth to the touch. Slag and expanded clay aggregate, because of their open texture and nonpolishing characteristic, provide an excellent mechanism for drainage escape. Surfaces provide, in addition to water escape channels, an excellent means of reducing "ice glaze" potential, which is prevalent after freezing rain and snow. This was observed during January and February 1973 when Louisiana was experiencing unusual snow and ice conditions. The hot-mix section, because of its dense texture, had a heavy glaze over it that made driving hazardous.

The finished texture of these PMS surfaces provides a pleasing appearance and a smooth riding surface. Figure 5 shows a series of photographs of the surface textures of the various surfaces discussed. The roughness of these surfaces, as measured by the BPR roughometer after 12 months of traffic exposure, ranged from 97 in./mile (1531 mm/km) for hot-mix surfaces to 86 in./mile (1357 mm/km), 81 in./mile (1278 mm/km), and 76 in./mile (1199 mm/km) for gravel PMS, expanded clay PMS, and slag PMS surfaces, respectively.

Because higher than normal asphalt contents are required for PMS surfaces, the problem of bleeding, due to either additional compaction or particle loss, cannot be ignored because it tends to induce plastic smoothing of the surface, thereby causing loss

Table 4. Summary of frictional performance, interior lane, low traffic volume.

Age (months)	Cumulative Traffic (millions of vehicles)	Skid Numbers at 40 mph by Type of Surface			
		Gravel Sand Hot Mix	Gravel PMS	Expanded Clay PMS	Slag PMS
0	0	44	43	57	53
4	0.46	50	52	66	59
12	1.42	51	52	66	61
18	2.13	52	49	61	57
32	2.68	48	54	60	55
40	3.34	50	47	66	56
48	3.98	45	44	61	58

Note: 1 mile = 1.6 km.

Figure 2. Skid performance of various PMS and hot-mix surfaces, exterior lane, high traffic volume.

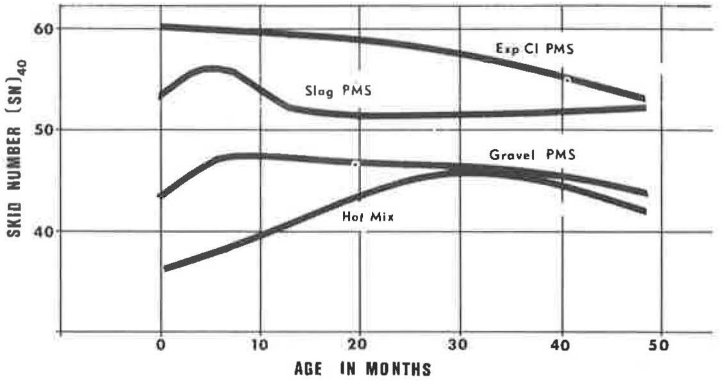


Figure 3. Skid performance of various PMS and hot-mix surfaces, interior lane, low traffic volume.

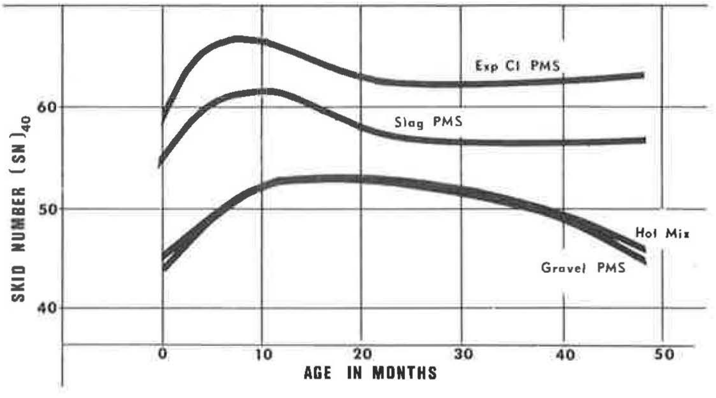
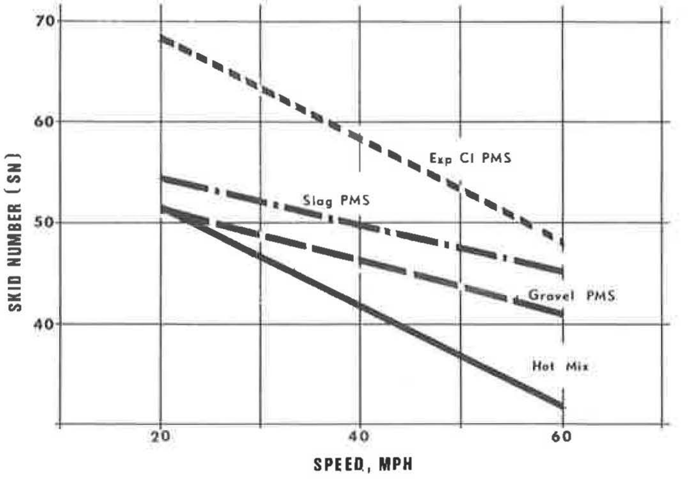


Figure 4. Friction-speed relationship of various PMS and hot-mix surfaces.



of texture and, ultimately, reduction in SN. However, this condition was not observed for any of the surfaces, except expanded clay, during the 4 summers that the surfaces were exposed. The initial low SN values observed soon after resurfacing can be attributed to thick asphalt films on protruding aggregate particles. This film soon wears off, and the full skid resistance of the surface is restored.

PRESENT CONSIDERATIONS

Since the construction of these PMS surfaces and because of their favorable skid performance, more PMS surfaces have been constructed. So far, the performance of these sections has been good.

Slag PMS

A slag PMS surface was constructed on a 0.846-mile (1.362-km) segment of I-10 in the New Orleans area in June 1970. Gradation and asphalt content were basically the same as for the slag PMS used on the experimental section (Table 1). The 6-lane segment of the highway carries more than 75,000 vehicles per day. The old surface had SN values in the low 30s and had a high percentage of wet accidents.

Figure 6 shows the skid performance of the 6-lane divided highway over a 30-month period. High initial SNs and subsequent uniformity are evident from these figures. However, the major benefit from the thrust behind the resurfacing is given in Table 5, which lists the accident data before and after corrective measures were taken. Accident data are given for each 6-month period before and after construction. The wet accident rate was computed on the basis of the number of vehicles exposed during wet conditions. Average hourly traffic and rain factor were used to compute exposure rate. The reduction in wet accidents is quite significant.

The microtexture and macrotexture of this PMS surface are such that during rainy weather the surface does not have the wet, shiny look generally associated with dense-graded asphalt concrete surfaces. And, bleeding due to high summer temperatures and traffic has not been observed in wheel paths.

Granite (Nepheline Syenite) PMS

This was the first use of the granite material in Louisiana. Two half-mile sections were constructed in January 1973, on segments of US-80, a divided 4-lane highway. The sections were constructed on westbound lanes that had an ADT of 10,080. The primary purpose for resurfacing was to provide a mechanism for water escape and reduce the hydroplaning potential rather than to increase the SNs that were in the upper 30s before the resurfacing. Although sufficient time has not elapsed to evaluate skid performance, 6-month SN data are in the middle 40s. The resurfacing has reduced the hydroplaning potential by providing easy drainage channels.

FUTURE CONSIDERATIONS

On the basis of data on various PMS surfaces, Louisiana has decided to made extensive use of slag and expanded clay aggregate PMS surfaces on all asphalt concrete roadways under heavy traffic (over 4,000 ADT/lane) (4). Supplementary specifications were prepared to cover design, construction, and acceptance procedures for these PMS surfaces.[1] Present policy also will allow gravel PMS on roadways having ADT between 1,000 and 2,000. Furthermore, during cool weather, the asphalt may be heated to 280 F (137.8 C) to allow for proper coating.

Design

At present, determining optimum asphalt content is based on visual observation of

[1] The original manuscript of this paper included the Appendix, Supplemental Specifications. The Appendix is available in Xerox form at cost of reproduction and handling from the Transportation Research Board. When ordering, refer to XS-53, Transportation Research Record 523.

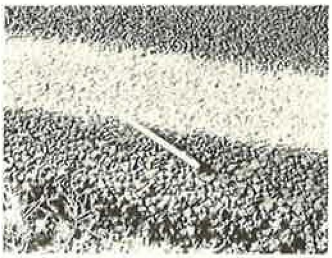
Figure 5. Textures of various surfaces.

dense-graded hot mix

expanded clay PMS

slag PMS

95% crushed gravel PMS



granite PMS (U.S. 80)

Figure 6. Skid performance of slag PMS surface.

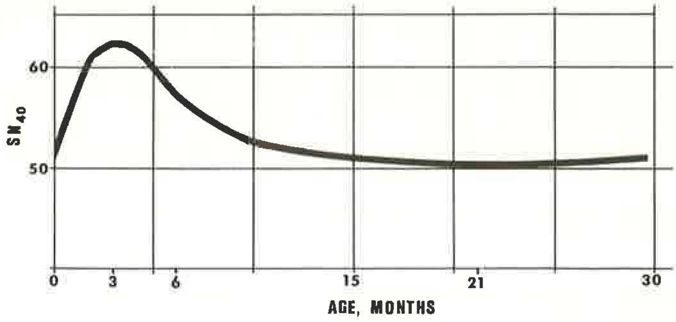


Table 5. Accident reduction before and after resurfacing.

Period	Accidents Total	Accidents Wet	Accidents Percent Wet	Rain Factor[a]	Accident Rate Total (mvm)	Accident Rate Wet	Accident Rate Reduction Total (percent)	Accident Rate Reduction Wet
Before								
January 1970 to May 1970	37	21	56.7	70	3.78	114.1		
After								
June 1970 to December 1970	38	9	21.0	84	2.80	41.1	25.9	64.0
December 1970 to June 1971	17	2	11.8	29	1.72	28.6	54.5	74.9
June 1971 to December 1971	38	5	31.2	96	2.78	19.2	26.5	83.2

[a]Number of 1-hour periods during which 0.1 in. (2.5 mm) or more rain was recorded.

various asphalt content mixes and the boil test for stripping characteristic of aggregate asphalt mixes. Fifty-blow Marshall specimens are sometimes prepared to determine the breakage resistance of the aggregate particles. The drainage characteristic of a mix is determined by an outflow meter closely matching the device described by Brakey (5). In addition, the specimens are sawed in half for visual examination. The final recommendation on the job mix is made from the evaluation of the data. With the application of these design procedures, it has been possible to reduce asphalt content to 6.75 percent and 13.0 percent for slag and expanded clay aggregate PMS mixes respectively.

Construction

Although preparation of PMS mixes at the hot-mix plant is similar to the preparation of conventional mixes during construction, the mixing temperature should not exceed 260 F (126.7 C). Long hauls of more than 40 miles should be avoided or excessive "slumping" and separation may result. Any hand placement or raking tends to destroy uniform texture. This was observed on the granite PMS project in which the contractor had little choice at turnouts and connections; the texture at such locations was uneven and easily could be delineated from adjacent sections. A uniform tack coat is a must. In most cases, only a tandem roller is needed for compaction. And, because of the open-graded texture of these mixes, a means for preventing asphalt stripping should be provided.

These PMS surfaces should never be used without dense surface under them; otherwise, water will penetrate into the base and create adverse conditions due to their saturation. Drainage also should be provided when these mixes are placed in curb and gutter sections. If paved shoulders are used, the PMS surfaces should be higher than the adjoining shoulder.

Cost

Initial cost evaluation of plant-mix seals gave figures slightly higher than for conventional mixes—\$0.60 to \$0.65/sq yd for a $\frac{5}{8}$-in.-thick surface (\$0.72 to \$0.78/m^2 for a 16-mm-thick surface). However, these materials, because of differing unit weights, will provide different coverage for a given thickness. For example, if gravel will cover 1.0 sq yd (0.86 m^2), an equal weight of slag will cover 1.2 sq yd (1.00 m^2) and expanded clay will cover 2.2 sq yd (1.84 m^2).

SUMMARY

Plant-mix seals seem to be the most promising resurfacing mechanism because they provide higher initial SNs, desired uniformity, and flat friction-speed gradients. An acceptable level of skid resistance can be maintained successfully if nonpolishing aggregates are used and if the treatment is applied carefully. In addition to providing high skid resistance, the surfaces also reduce the potential for hydroplaning. The safety features, coupled with a pleasing appearance, smooth, quiet riding surface, and reasonable cost, should make them the front-runners of available resurfacing methods.

ACKNOWLEDGMENTS

The findings reported in this paper were obtained under the Louisiana HPR Program in cooperation with the Federal Highway Administration. The assistance of W. C. Walters, S. G. Bokun, and J. D. Bennett in providing data on skid resistance is gratefully acknowledged. The opinions, findings, and conclusions expressed in this publication reflect the views of the authors and not necessarily those of the Federal Highway Administration.

REFERENCES

1. Kummer, H. W., and Meyer, W. E. Tentative Skid Resistance Requirements for Main Rural Highways. NCHRP Rept. 37, 1967.

2. Mills, J. A., III. A Skid Resistance Study in Four Western States. HRB Spec. Rept. 101, 1969.
3. Skid Resistance. NCHRP Synthesis of Highway Practice Rept. 14, 1972.
4. Interim Policy—Use of Plant Mix Seals. Memorandum from Chief Engineer, Louisiana Department of Highways, March 26, 1973.
5. Brakey, B. A. Design, Construction and Performance of Plant Mix Seals. Paper presented at 55th Annual Meeting of AASHO, Phoenix, Arizona, 1972.

CONVENTIONAL CHIP SEALS AS CORRECTIVE MEASURES FOR IMPROVED SKID RESISTANCE

Bob M. Gallaway and Jon A. Epps, Texas Transportation Institute,
Texas A&M University

Chip seals are used to improve the surface friction or skid resistance of streets and highways. Their desirability is discussed. Properties including aggregate gradation, type, size, and mineralogy and surface texture are reviewed; bituminous binder type, viscosity, and amount are discussed and related to field experience. Relations of factors associated with the binder and the aggregate are evaluated. Also evaluated are design, construction, and performance to improve skid resistance of the finished surface.

•THE LITERATURE abounds with articles dealing with the many facets of street and highway renovation or improvement by conventional chip seal, which consists of separate applications of bituminous binder and cover aggregate. Chip seals have for many decades been used primarily for purposes other than improved skid resistance although improved skid resistance would often result from this type of maintenance. In this paper attention is centered on chip seals used as corrective measures for streets and highways with undesirably low surface friction or skid resistance. Pros and cons from the owner-user and producer-contractor viewpoints are discussed. Basic factors such as material properties including aggregate gradation, type, size, and mineralogy; surface texture and size; and bituminous binder type, viscosity, and amount are related to field experience as these factors affect the skid resistance properties of various material combinations under traffic in rural and urban areas.

Past investigations have dealt with basic objectives and benefits of conventional seal coats (2, 3, 4). Researchers have reported on design procedures, aggregate requirements, and construction-related operations (1, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14). More recently, however, investigators have directed increasing attention to the skid resistance properties of seal coats and the desirable attributes of cover aggregate and bituminous binder (15, 16, 17, 18, 19, 20).

Kari, Coyne, and McCoy (21), who described in detail the relationship of the input of the binder to the success of the job, dealt with the desirable properties of binder consistency and durability. Specifically the authors stated that

> Asphalt binders suitable for seal coats must have the following properties:
>
> 1. Be capable of being sprayed uniformly over the road surface. Streaking, bleeding and raveling can be minimized by controlling the uniformity of longitudinal and transverse spread.
> 2. Resist runoff, i.e., not flow off the pavement after application. This insures sufficient binder to prevent loss of cover aggregate on grades and super elevations.
> 3. Wet the aggregate and be sufficiently fluid to permit compaction of the seal. This becomes critical in cold weather due to the increase in asphalt viscosity.
> 4. Rapidly develop cohesion and bond to both the pavement and mineral aggregate. Shoving and scuffing during hot weather will result when cohesion is low.
> 5. Resist displacement by water and other disruptive forces (i.e., gravitational and mechanical forces). Loss of bond between the asphalt and aggregate will result in raveling.

Publication of this paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction.

6. Resist factors influencing aging. Hardening of the binder will result in fracture of the asphalt film and raveling under traffic.

7. Be uniform from one delivery to the next. Product uniformity aids in the successful construction of a seal coat. Product uniformity is dependent upon suitable specifications. All of the desired performance properties listed above are related to or can be described in terms of consistency and durability.

One may readily infer from this list of requirements that a relationship among weather, climate, and binder properties exists and that binder durability is vital. Aggregate properties that are necessary for producing high-quality seal coats include items such as amount of stone, gradation, size, shape, abrasion resistance, color, moisture condition, cleanliness, adhesion, freeze-thaw resistance, and polish susceptibility. Details of the relative effects of these properties are discussed by Herrin, Marek, and Majidzadeh (1); Kersten and Skok (11); McLeod (2); Gallaway and Harper (12); Benson and Gallaway (5); and Wilson (22).

Each method on the design of seal coats contains certain differences based primarily on available materials, individualized traffic demands and to some extent the personal likes and dislikes of the person who developed the method. Design methods deal primarily with application rates for binder and cover aggregate with estimated adjustments for condition of the surface to be sealed, amount and type of traffic expected during the estimated life of the seal, and climatological effects of these factors.

Hveem, Lovering, and Sherman (9) proceeded from the work of Hanson (23) and developed nomograms to estimate amounts of binder and cover stone for given material properties and traffic demands. Nevitt (24), in his work on seal-coat design, stressed a point that has grown continuously in importance over the years—the thrifty use of all materials and efforts. Nevitt also stressed the importance of aesthetics, a point that commands the respect of concerned road maintenance personnel today. Others who have published seal-coat design procedures are Kearby (6), Benson and Gallaway (5), Lovering (25), Kuipers (26), and McLeod (2). Their common design thread is the selection of the proper amount of binder for a given top size and grading of cover stone. Usually a binder quantity adjustment for road surface condition is included. Primarily, this adjustment is based on surface texture, although none of the articles specifically referred to adjustment as being based on surface texture.

Two factors omitted by many writers are those of traffic type, volume, and weight and effect of a soft substrate. Nor was much said about climate in relation to selection of binder viscosity.

Empirical curves were presented by Gallaway (27) for estimating a binder quantity correction for traffic volume. These curves assume average rural traffic, 15 percent of which is trucks. The correction provided for additional binder scaled from no correction for traffic above 1,600 to 2,000 vehicles per day (vpd) for 2 lanes to a maximum correction of 0.05 to 0.06 gal/sq yd (0.23 to 0.27 litre/m^2) for traffic volumes less than about 50 vpd.

Cover aggregate may be submerged in the binder, not because the design was incorrect but because the stone was forced into the underlying substrate or existing surface. Problems of this type are associated with inadequate compaction (low density) of the surface layer that is to receive a seal. It is often relegated to restricted areas such as patches that have been made before sealing. If parts of the surface require reworking in preparation for sealing, they must be adequately dense to prevent intrusion of the cover stone because intrusion often results in flushing.

For an average rural highway carrying a traffic volume of less than about 2,000 vpd, the percentage of trucks may be expected to be rather low and restricted in loads; therefore, the usual seal coat design will disregard the effect of heavy loads and high tire pressures. The ill effects of this omission are shown in Figure 1. In the construction of this seal coat a full road width distributor was used to apply binder to the entire surface in 1 pass. Loaded haul trucks, not anticipated in the design stage, caused the problem in the flushed lane. The other lane, which apparently has received the design amount of traffic, is performing beautifully.

Excessive horizontal shear forces can cause similar problems; these will exist on

Figure 1. Flushing seal coat caused by unexpected truck traffic.

Figure 2. Flushed seal coat caused by cornering action of traffic dislodging cover stone.

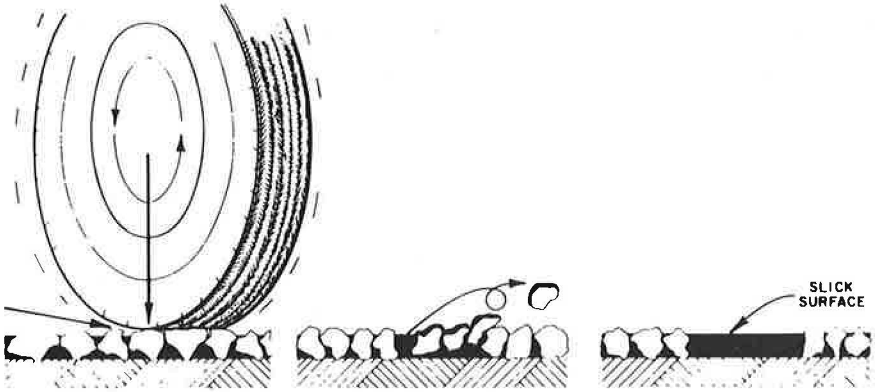


Figure 3. Binder demand affected by surface hunger.

Figure 4. Lack of uniformity of surface to be sealed.

sharp curves and at intersections, particularly, in urban areas. A schematic of this effect is shown in Figure 2. Different approaches may be used to solve this problem. One is to avoid the use of a seal when it would be subjected to this type of traffic. Another possibility would be to select a higher viscosity binder and use this in combination with a smaller-sized cover stone. A smooth-textured stone would not be dislodged as easily as a rough-textured stone, but for safety reasons a low-friction surface should be avoided.

MATERIAL SELECTION

In preparing specifications for a job, one should remember to write them around (a) available materials, (b) contractor capabilities, (c) buyer's willingness to accept contractor's finished product, and (d) for most public streets and highways, general public acceptance of facility performance. Selection of the binder and cover aggregate must be economically justifiable as well as technically sound (25). The general characteristics of the binder have been discussed in detail by many writers (2, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38). Much also has been written on the interaction of the properties of cover aggregate and binder for a given design and environment and how it relates to skid resistance of the surface (18, 19, 20, 39, 40, 41, 42, 43, 44, 45, 46, 47). It is evident from the findings that for prolonged high skid resistance, the selected cover aggregate must possess and maintain both macrotexture and microtexture during its service life. Adequate macrotexture may be available in suitable size ranges in both natural and manufactured aggregates.

Natural aggregates that abrade rather than polish under the action of weather and traffic usually meet microtexture requirements as do aggregates composed of a proper mixture of hard and soft particles. Sandstones are examples of the former and conglomerates, the latter. Hard particles dispersed in a soft matrix such as silica sand in a limestone matrix have also been found suitable as nonskid cover stone (19, 48).

Lightweight manufactured aggregates have been used widely to produce high-friction surfaces. The critical property of such materials is microtexture, which exists throughout individual "stones" as blebs or gas pockets formed during the heat cycle of manufacture. Such microtexture is subject to continuous renewal under the action of traffic. The manufacture of "engineered" aggregates is technically and economically feasible, and proof of performance has been published by James (49), Britton (45), and Gallaway and Epps (42). Again, a key property is that of renewable microtexture, which is often controllable in raw material formulation and in manufacturing.

A factor of primary importance that is often entirely neglected in the design phase is the magnitude of the tumbling force of a pneumatic tire operating in the cornering slip mode, a common mode in urban traffic. The magnitude of this tumbling force is affected primarily by the friction between the tire and the contacted aggregate and the length of the moment arm from the center of rotation of a given stone. This moment arm, therefore, directly relates to the size of the cover aggregate. Or, large stones generally are more easily tumbled than small ones. This assumes roughly equal bond tenacity for all sizes of stone, which seldom prevails because an adequate design calls for a binder quantity equivalent to an embedment depth equal to about half the average stone size. So, a dilemma exists. It is questionable whether conventional seal coats should be used in moderate to heavy urban traffic for this reason. The reasoning is sound, and we recommend against the use of seals under this type of traffic. The use of high-friction aggregate simply aggravates the problem. If one persists in the use of chip seals for medium to heavy urban traffic, extreme care in the design and construction phases of the job is absolutely necessary. Things to be considered include condition of the surface to be sealed (Figs. 3 and 4), type and weight of traffic to be encountered, and climate.

Conventional seal coats are highly effective and economically justified on lightly traveled city streets, county roads, and rural highways subject to traffic volumes up to approximately 4,000 vehicles per lane per day, provided such rural highways do not have numerous steep grades and sharp turns. This type of road would be in the same category as those that carry heavy urban traffic.

Use of single-sized aggregates is logical because one can closely determine binder demand for an assumed embedment depth. The margin for error in arriving at the design binder application rate is greater for single-sized stone. Specifying an aggregate size is simple, but its economical production is often difficult. For some sizes for which there are limited needs, prices are high. Handling and hauling may cause the specified sizes to change and cause rejection at the job site. Although seals made from such select stone are visually pleasing, it is difficult to attribute service performance to stone size alone. One might claim that a certain stone size causes better water escape at the tire-pavement interface at high speeds and under inclement weather conditions, but this would be difficult to prove.

Carefully controlled laboratory tests by Benson and Gallaway (5) and others have shown conclusively that cover aggregates with excess fines cause extensive problems. Because such cover aggregates are usually available at reduced costs, they are used when price is important in material selection. But, results are often disastrous especially when uniformly good skid resistance is desired. Size control is definitely important to uniform skid performance of a road surface. Macrotexture obtained by exposed rugosity of cover aggregate ensures water escape at the tire-pavement interface and in locked-wheel stops. So, a compromise is necessary. Small amounts of oversized and undersized material can be permitted in specifications. However, caution is recommended, particularly on the permissible amount of dust. More than 1 percent dust is highly detrimental to the early establishment of a bond. A disadvantage of oversized material is dislodgment by traffic, which causes flying stones (12). A wide range of sizes makes it difficult to optimize binder quantity. Small particles will be inundated and large stones will be embedded too lightly. Oversized material should not exceed about 2 percent and no material should be retained on the next larger sieve. Example specifications of the Texas Highway Department (THD) are given in Table 1 for item 302, class B cover stone (50). Sizing requirements for THD item 303, lightweight aggregates, are given in Table 2. It is interesting to note that similar grades given in these tables are not sized the same way. The lightweight material is somewhat coarse probably because some breakdown is expected in hauling and handling. And, only 3 grades (size ranges) are given for the lightweight material. Extensive experience in Texas with lightweight manufactured aggregates has shown conclusively that grades other than these 3 are both unnecessary and undesirable. Larger sizes are generally more difficult to produce and are usually structurally weak; smaller sizes create design and construction control problems.

Bituminous binders for seal coats include asphalts and tars, and both materials have performance advantages. The primary differences in the 2 types of binder are temperature susceptibility and wetting ability. Tars are more susceptible to change in viscosity with change in temperature, and they are better wetting agents than are asphalts. Both binders are available in different forms such as cements, cutbacks, and emulsions and in a wide range of viscosities. This wide choice of binder form and property adds to the difficulties of the seal-coat designer. Uniform distribution in the desired amount of residual cement is the general objective in seal-coat work, but performance viscosity is often critically important. Selection of the form of binder may be based on convenience. Let us assume that different forms and viscosities are available and that recommendations will be made in keeping with construction constraints and service demands for improved surface friction. Binder service viscosity should be determined by compromise, considering the primary factors of climate, condition of surface to be sealed, and anticipated traffic including both volume and weight. Other minor factors may be included, but generally these will be found to have an effect so small as to be clouded by lack of construction control. Cold climates require softer binders with viscosities around 300 stokes (0.03 m^2/s) at 60 C (THD AC-3); hot climates require hard binders with viscosities of 1,000 to 1,500 stokes (0.1 to 0.15 m^2/s) at 60 C.

Bond tenacity is critical and is determined primarily by binder viscosity at service temperatures and aggregate surface properties and secondarily by depth of stone embedment (51). Establishment of this bond is assumed to be effected by intrusion of the stone into the binder and its preferential wetting (29). Wetting of the stone is enhanced by having the stone clean and dry for hot cements and cutbacks and clean and slightly

Table 1. Class B aggregate for surface treatments.

Sieve Size	Percent Retained Grade 1	Grade 2	Grade 3	Grade 4	Grade 5
1 in.	0	0	0	0	0
7/8 in.	0 to 2	0	0	0	0
3/4 in.	20 to 35	0 to 2	0	0	0
5/8 in.	85 to 100	20 to 35	0 to 2	0	0
1/2 in.	—	85 to 100	20 to 35	0 to 2	0
3/8 in.	95 to 100	95 to 100	85 to 100	20 to 35	0
1/4 in.	—	—	95 to 100	—	0 to 5
No. 4	—	—	—	95 to 100	—
No. 10	99 to 100	99 to 100	99 to 100	99 to 100	99 to 100

Note: 1 in. = 25.4 mm.

Table 2. Lightweight aggregate for surface treatments.

Sieve Size	Percent Retained Grade 3	Grade 4	Grade 5
3/4 in.	0	0	0
5/8 in.	0 to 5	0	0
1/2 in.	30 to 50	0 to 5	0
3/8 in.	85 to 100	20 to 40	0 to 2
1/4 in.	95 to 100	—	—
No. 4	—	95 to 100	60 to 80
No. 10	98 to 100	98 to 100	98 to 100

Note: 1 in. = 25.4 mm.

Figure 5. Shaded areas of seals using emulsions develop bond more slowly than exposed areas.

Figure 6. Binder demand may vary across pavement and transverse adjustment of spray bar output may be required.

wet for emulsions. Wetting of the stone by the binder requires time. And more time is necessary when viscosity increases at the time the 2 materials are mated. For example, bituminous cements sprayed at 300 F (150 C) on a road surface at 120 F (40 C) will cool to approximately 130 F (54 C) in less than 3 min. Cover stone is seldom applied within this time, and, therefore, one should not assume that the binder is liquid when the stone hits it. Time and force is required for intrusion and wetting. Stone that is wet or dirty or both impairs the wetting rate; these adverse factors must be considered to arrive at the delay time before traffic is allowed on the surface. Emulsions have the advantages of easy intrusion and quick wetting. They also are nonpollutants. McKesson (8) in 1948 reported on the use of emulsions for seals as did Bower (36) in 1960 and Bohn (37) in 1963. Recent improvements in the uniformity and quality control of emulsions plus the adverse effect of cutback on the environment should lead to a continued increase in the demand for emulsions. Quick setting cationic emulsions made from high-viscosity binders are most effective in warmer climates. Lower viscosity base cements should be used in cold climates. A definite advantage of the cationic type is that weather has minor effect. According to J. Dybalski of the Armak Chemical Company such emulsions break primarily by surface attraction and can be formulated for controlled break rates even under conditions of high humidity and cool weather.

In constructing seals that use emulsions, the use of pilot cars is advised for traffic control. Before allowing traffic on a newly sealed surface, a check for degree of break and bond tenacity should be made in shaded areas of the road surface. Break is usually delayed in such areas, and, if the road is turned over to traffic prematurely, excessive whip-off may result in shaded areas (Fig. 5). This problem is associated particularly with anionic emulsions, which break by evaporation or absorption or both. Cationic emulsions are affected only slightly. The shade effect is generally more prevalent in residential areas of cities than elsewhere. Cutback asphalts have been used worldwide as binders for seals for decades (52, 53, 54, 55). The use of cutbacks in cold climates is technically valid, but even in these areas flushing may be a problem during the summer months, according to Robinson (52). As an expedient, cutbacks have been and are being used in the fall in Texas, but bleeding during early summer of the next year is a common fault and makes their use suspect for most other reasons. Two alternate solutions are suggested:

1. Use hot cement and heated cover stone, or
2. Use a cationic emulsion with heated stone, if necessary.

Early establishment of bond is enhanced by lightly precoating the aggregate with 0.5 to 1.0 percent of a medium curing type of cutback. Precoating of cover stone is widely practiced with generally improved results over natural aggregates, at least for early establishment of bond. There is little evidence that friction values are changed appreciably by precoating.

EQUIPMENT AND CONSTRUCTION OPERATIONS

Seal coats are often used on roads that should be completely reconstructed. Many times the results of such a decision are embarrassing. If a surface to be sealed is in need of spot improvement to restore riding quality and improve structural capacity, this should be done 60 to 90 ninety days in advance of the seal-coating operation. Such planned repairs improve the probability of a successful seal. Herrin, Marek, and Majidzadeh (1) made sound recommendations concerning the preparation of the underlying surface. Power brooming is a necessary prelude to binder application. In urban areas a vacuum attachment to the broom is advised and some manual cleaning may be necessary.

Before applying the binder to the throroughly cleaned surface the pressure distributor should be calibrated and checked for operational efficiency. Schuelie (14) has reported on equipment needs for seal-coat work and emphasizes the need for spray bar calibration. Selected districts of the THD require a complete annual recalibration of distributors used in those districts. Additional check tests are made at tne beginning of each day's operation. Because of variations in binder demand across lanes of some pavements, it is often advisable to reduce binder application in the wheel path. An example of variable transverse demand is shown in Figure 6. Reduced application of binder is accomplished by substituting smaller nozzles in the spray bar at the point of reduced demand.

Uniform spreading of the design amount of stone is made easier with a continuous feed machine as opposed to a tailgate spreader. The use of a deflector to aid in the separation and earlier application of the coarser fraction of the cover stone ensures a better job. Some spreader operators remove this deflector to make their job easier, but this is not advised. This deflector is designed to assist in the uniform application of the aggregate. Cover stone application rates are extremely difficult to estimate if the judgment is made immediately behind the spreader. A more reliable approach is to determine by laboratory tests on stockpiled aggregate the amount of stone required to cover a unit area to a single stone depth. This quantity is then translated into field units of square yards of surface per cubic yard of stone. For example, the grade 4 lightweight aggregate of THD item 303 given in Table 2 would require about 9.5 lb/sq yd (5.1 kg/m^2) [based on an assumed unit weight of 50 pcf (800 kg/m^3)] or a field cover rate of about 140 sq yd/cu yd (155 m^2/m^3). Natural rounded gravel, graded as given in Table 1 and weighing about 95 pcf (1 520 kg/m^3), would cover an equivalent area with a unit cover rate of about 18 lb/sq yd (9.8 kg/m^2). Technicans determining laboratory cover rates for the first time will usually err on the high side, often by as much as 15 to 20 percent. This can result in expensive field mistakes. To avoid such mistakes the technician should strive toward a minimum amount of stone to cover a unit area. A convenient unit to use in the laboratory is ½ sq yd or ½ m^2. A suggested approach for a novice is to carefully cover the unit area with what appears to be sufficient stone. Determine the amount used by weighing the stone. In sequential steps remove 5 percent increments and rearrange the remaining stone each time until it is apparent that the unit area is not adequately covered. At this point return one of the 5 percent increments to the surface. This amount should be close to the quantity determined by an experienced operator.

Why all the fuss about minimizing the stone cover rate? First, it is wasteful to apply excess stone. Second, normal-weight stone left loose on the pavement surface is hazardous. Loose stone may be thrown by traffic and cause windshield and headlamp damage. Excess loose stone contributes to low friction on an otherwise safe surface. Third, excess stone contributes to crushing and dusting, both of which are undesirable. Dusting is a traffic hazard in the early use of the road; crushing disturbs a balanced design of stone size and binder quantity, which can result in flushed areas. Naturally, crushing is more of a problem for seals over rigid pavements than it is for seals on flexible surfaces (56). Let us assume that we have applied the selected binder and cover stone at the proper rates and we can turn to the rolling operation. Rolling of cover stone is a necessary and important step in the successful construction of a nonskid seal coat. Although under certain circumstances a flat-wheel steel roller may be used on seals, their general use is not advised. Self-propelled pneumatic rollers equipped with smooth tires inflated to about 30 to 45 psi (207 to 310 kPa) are highly recommended for the seating of cover stone in seal-coat operations. This type of roller is more effective for 2 major reasons, because the kneading action of this type of roller does a better job of fitting the aggregate into a continuous mosaic than does a steel roller and because crushing is reduced substantially. Pneumatic rolling requirements for seals fall in the range of 5 to 7 hours/mile (3 to 4 h/km) of 2-lane highway. Roller speeds in excess of about 7 mph (11 km/h) are not recommended.

After the rolling operation and a delay period of 25 to 48 hours the sealed surface should be lightly broomed to remove any loose stone. Required lane marking of the finished surface should follow as soon as is practical.

LEVELS OF PERFORMANCE

The level of performance that may be expected for a properly designed and constructed seal coat using cover aggregate sized from ½-in. to No. 4 sieve size is shown in Figure 7. The superior skid performance indicated for stone with adequate microtexture is emphasized. Also, it is suggested from Figure 7 that where polishing is in evidence the surface is speed sensitive under wet conditions. Figure 8 treats the effect of increasing volumes of traffic on skid numbers at 40 mph. Polish-susceptible cover stone is undesirable for heavy volumes of traffic.

The expected improvement of existing, dangerously slick surfaces is shown in Figure 9. It is assumed that the improvement results from proper design and construction with cover aggregate in the range of ½-in. to No. 4 sieve size.

SPECIAL PROPERTIES OF COVER AGGREGATE

Special properties of particles—shape, surface texture, durability, and polish susceptibility—as they relate to improved skid resistance of a pavement surface, warrant further attention.

Schonfeld (57), Gallaway and Rose (44), Britton (45), Britton and Gallaway (43), Gallaway, Schiller, and Rose (58), and Gallaway, Rose, and Hutchinson (41) have recently dealt extensively with particle shape, particle surface texture, and pavement macrotexture and microtexture.

It is apparent that estimating friction by Schonfeld's method must, in the case of seals, rely on the coarse aggregate and its size and surface properties (57). Gallaway and Rose (44) in their measurements of the macrotexture of typical Texas highways found that macrotexture related well to friction for an aggregate of a given source and that, for constant microtexture, macrotexture had a primary effect on friction gradient or change in friction with speed. Tabor (59), in his study of hysteresis, emphasized the importance of macrotexture's effect on frictional drag in locked-wheel stops. Van der Burgh and Obertop (60) reported on size and shape of road surface projections and detailed the relationship between various types of rubber and road surface macrotexture. Macrotexture depth and spacing were key parameters.

Britton (45) developed a master curve for the adhesion component of wet tire-pavement surface friction in which a reduced friction number was plotted against a reduced microtexture size parameter. It is evident from this and other reported findings that microtexture has a most important effect on pavement friction. It is further evident from Britton's work that as microtexture size is increased wet friction increases, reaches a maximum value, and then decreases. He also demonstrated that it is technically feasible to produce synthetic aggregates with controlled microtexture.

Currently, many U.S. highways and city streets are being surfaced with commercial grades of lightweight synthetic aggregates. Such aggregates are made primarily by the rotary kiln process from clays or shales. Microtexture of a desirable size range is a property of these materials.

The desirable aggregate property of durability also has its disadvantages because a very hard material, although slow to polish, will polish more than a less durable material. Generally it is necessary to crush such material to maximize the hardness advantage. Aggregates composed naturally of both hard and soft materials therefore have nonpolishing properties. For such materials the attrition of traffic furnishes a surface friction renewal mechanism. Crushing costs of such materials are usually minimal.

Polish susceptibility of aggregates is related primarily to type of mineral and purity. Type of mineral and purity strongly affect hardness and toughness, and these properties in turn determine crushing costs. Granites, traprock, and limestones make up much of the crushed stone used in the United States; these materials vary greatly in polish susceptibility with limestones considered most susceptible, particularly those of high purity and fine-grain structure.

Laboratory and pilot scale tests have been developed to measure polish susceptibility (61, 62, 63). Values coming out of these tests are used in specifying polish resistance of cover stone. One should select the restrictive values with care and take into account service demands, weather, pavement geometrics, terrain, and long-term total costs. Polish-susceptible limestones are entirely satisfactory as cover stone provided traffic volume and weights will not polish the stones to an unacceptably low value. The key to

Figure 7. High-speed performance of typical seal coats under inclement weather conditions.

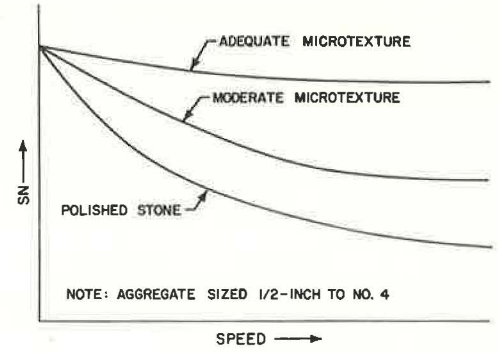


Figure 8. Typical performance of various types of cover aggregates used on conventional seal coats.

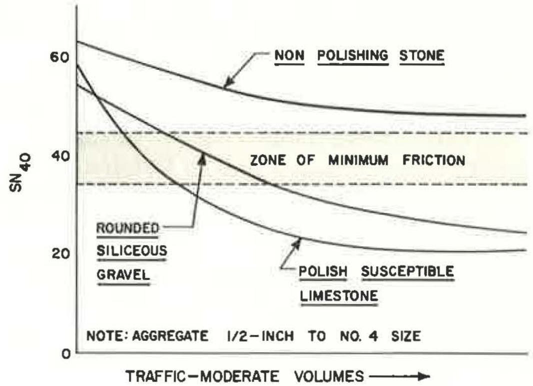


Figure 9. Expected average improvement of polished road surfaces after seal coating with selected aggregates.

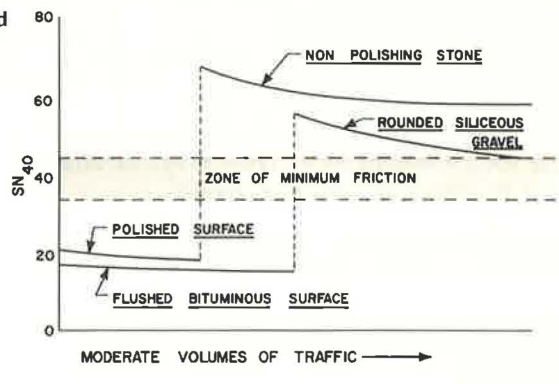
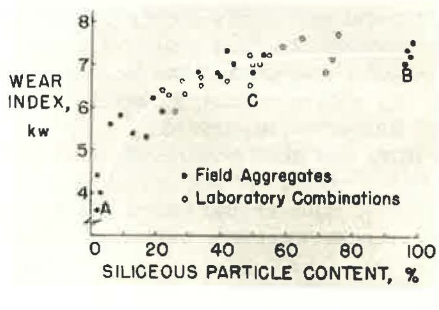


Figure 10. Limestone with adequate silica inclusions produces skid-resistant surfaces with proper design and construction procedures.



the success of polish-susceptible limestone under such service is continued chemical reaction between the stone and the air. This reaction causes microtexture renewal. As a rough rule of thumb, traffic of less than 250 vehicles per lane per day can be tolerated without excessive polishing. When available in sufficient amounts, properly sized silica impurities in limestone ensure nonskid properties. Field confirmation of the serviceability of this type of impure limestone has been common (48). A relationship between wear index (friction) and silica content is shown in Figure 10 (64).

Aside from these special properties of cover aggregate, the importance of good construction control cannot be overemphasized. The very best materials, design, and equipment used under ideal weather conditions can result in miserable failures when skill and pride in construction are neglected.

REFERENCES

1. Herrin, M., Marek, C. R., and Majidzadeh, K. State of the Art: Surface Treatments, Summary of Existing Literature. HRB Spec. Rept. 96, 1968.
2. McLeod, N. W. Basic Principles for the Design and Construction of Seal Coats and Surface Treatments With Cutback Asphalts and Asphalt Cements. Proc. AAPT, Vol. 29, Supplement, 1960.
3. Sprayed Work. In Principles and Practice of Bituminous Surfacing, National Assn. Australian Road Authorities, Vol. 1, 1965.
4. Coulter, T. M. The Design of Bituminous Pavements. Main Roads, Vol. 24, No. 2, 1958.
5. Benson, F. J., and Gallaway, B. M. Retention of Coverstone by Asphalt Surface Treatments. Eng. Exp. Station, Texas A&M Univ., Bull. 133, 1953.
6. Kearby, J. P. Tests and Theories on Penetration Surfaces. HRB Proc., Vol. 32, 1953, pp. 232-237.
7. Kilfer, C. V. The Use and Abuse of Seal Coats. Crushed Stone Journal, Vol. 24, No. 2, June 1949.
8. McKesson, C. L. Fundamentals of Surface Treatment With Emulsified Asphalts. Proc. AAPT, Vol. 17, 1948.
9. Hveem, F. N., Lovering, W. R., and Sherman, G. B. The Design of Seal Coats and Surface Treatments. California Highways and Public Works, Vol. 28, Nos. 7-8, July-Aug. 1949.
10. Mackintosh, C. S. Rates of Spread and Spray in Bituminous Surface Dressing of Roads. Civil Engineer in South Africa, Vol. 3, Oct. 1961.
11. Kersten, M. S., and Skok, E. L., Jr. Criteria for Seal Coating Bituminous Surfaces. Minnesota Dept. of Highways Investigation, Interim Rept., No. 626, 1969.
12. Gallaway, B. M., and Harper, W. J. Laboratory and Field Evaluation of Lightweight Aggregates as Coverstone for Seal Coats and Surface Treatments. Highway Research Record 150, 1966, pp. 25-81.
13. Asphalt Surface Treatments and Asphalt Penetration Macadam. Asphalt Institute Manual Series No. 13, 2nd Ed., Nov. 1969.
14. Schuelie, W. H. Recent Developments in Equipments Used for Seal Coats or Surface Treatments of Existing Bituminous Surfaces. Proc. AAPT, Vol. 24, 1955.
15. Shattuck, C. L. Protective Seal Coats for Bituminous Pavements. Military Engineer, Vol. 49, No. 327, 1957.
16. Creamer, W. M., and Brown, R. E. Application of a New Non-Skid Surface Treatment on Connecticut State Highways. HRB Bull. 184, 1958, pp. 10-16.
17. Building Safer Highways With Slag. National Slag Association, Washington, D.C., Tech. Bull. No. 264, 1968.
18. Gallaway, B. M. Design and Construction of Full Scale Stopping Pads and Spin-Out Curves to Predetermined Friction Values. Jour. Materials, Vol. 5, No. 2, 1970.
19. Gallaway, B. M. Skid Resistance and Polishing Type Aggregates. Jour. Amer. Society Safety Engineers, Vol. 14, Sept. 1969.
20. Gallaway, B. M. Maintenance of Flexible Pavements for Improved Skid Resistance. Proc. Conf. on Skid Resistant Surface Course, Rept. FHWA-RDDP-10-4, Jan. 1973.

21. Kari, W. L., Coyne, L. D., and McCoy, P. E. Seal Coat Performance. Proc. AAPT, Vol. 31, 1962.
22. Wilson, D. S. An Experiment Comparing the Performance of Roadstones in Surface Dressing. Transport and Road Research Laboratory, Harmondsworth, England, RRL Rept. 46, 1966.
23. Hanson, F. M. Bituminous Surface Treatment of Rural Highways. Proc. New Zealand Society of Civil Engineers, Vol. 21, 1934-35.
24. Nevitt, H. G. Aggregate for Seal Coating. Proc. AAPT, Vol. 20, 1951.
25. Lovering, W. R. Seal Coats, Economic Usage. Western Construction, Vol. 29, No. 4, 1954.
26. Kuipers, J. P. Moderne Oppervlaktebehandeling. Wegen, Vol. 29, No. 5, 1955.
27. Gallaway, B. M. A Manual on the Use of Lightweight Aggregate in Flexible Pavement Systems. Expanded Clay, Shale and Slate Institute, 1969.
28. Benson, F. J., and Gallaway, B. M. Study of Some Variables Affecting Retention of Aggregate by Asphalt Surface Treatments. Roads and Streets, April 1954, pp. 113-122.
29. Dickenson, E. J. The Wetting of Air-Dry Stone by Bituminous Binders in the Road Surface Dressing Operation. Jour. Institute of Petroleum, Vol. 47, No. 445, 1961.
30. Mathews, D. H. Adhesion in Bituminous Road Materials: A Survey of Present Knowledge. Transport and Road Research Laboratory, Harmondsworth, England, Research Note RN/3151/DHM, 1958.
31. Mertens, E. W., Coyne, L. D., and Rogers, E. D. Cationic Asphalt Emulsions: Recommended Tests and Specifications. ASTM, STP 294.
32. Lovering, W. R. Selecting Asphalt Binder for Seal Coats. Western Construction, June 1958.
33. Alcoke, W. H. Seal Coats With Latex Tested. Roads and Streets, Vol. 98, July 1955.
34. Gallaway, B. M. Durability of Asphalt Cements Used in Surface Treatments. Proc. AAPT, Vol. 26, 1957.
35. The Report of the Road Research Board for 1957. Dept. Scientific and Industrial Research, England, 1957.
36. Bower, H. C. Cationic Asphalt-Emulsion for Surface Treatments. Roads and Engineering Construction, Vol. 98, No. 2, Feb. 1960.
37. Bohn, A. O. The Breaking of Asphalt Emulsions for Surface Treatments. Proc. AAPT, Vol. 32, 1963.
38. Major, N. G. Basis for Selection of Binders for Chip Sealing. New Zealand Engineer, Vol. 20, No. 12, Dec. 1965.
39. Gallaway, B. M., and Epps, J. A. Current Methods for Improved Tire-Pavement Interaction. Trans. Engineering Jour., 1972, pp. 915-921.
40. Gallaway, B. M. A Review of Current Methods for Producing Skid Resistant Pavements in the United States of America. Proc. AAPT, Vol. 42, 1973.
41. Gallaway, B. M., Rose, J. G., and Hutchinson, J. W. A Summary and Analysis of the Attributes of Methods of Surface Texture Measurements. ASTM, STP 530, 1972.
42. Gallaway, B. M., and Epps, J. A. Tailor-Made Aggregates for Prolonged High Skid Resistance on Modern Highways. Proc. 2nd Inter-Amer. Conf. on Materials Technology, Mexico City, Aug. 1970, pp. 90-99.
43. Britton, S. C., and Gallaway, B. M. Manual for the Construction and Maintenance of Standardized Skid Surfaces. Texas Transportation Institute, Texas A&M Univ., Tech. Rept. 797-4, Nov. 1972.
44. Gallaway, B. M., and Rose, J. G. Macro-Texture, Friction, Cross Slope, and Wheel Track Depression Measurements on 41 Typical Texas Highway Pavements. Texas Highway Dept., Res. Rept. 138-2, July 1971.
45. Britton, S. C. Standard Reference Pavement Surfaces for the Evaluation of Skid Test Equipment. Texas A&M Univ., PhD dissertation, 1972.
46. Csathy, T. I., Burnett, W. C., and Armstrong, M. D. State of the Art of Skid Resistance Research. HRB Spec. Rept. 95, 1968, pp. 34-48.

47. Shupe, J. W., and Lounsbury, R. W. Polishing Characteristics of Mineral Aggregates. Proc. 1st Internat. Skid Prevention Conf., 1959.
48. Gray, J. E., and Renninger, F. A. The Skid-Resistant Properties of Carbonate Aggregates. Highway Research Record 120, 1966, pp. 18-34.
49. James, J. G. Calcined Bauxite and Other Artificial, Polish Resistant Roadstones. Transport and Road Research Laboratory, Harmondsworth, England, RRL Rept. LR84, 1968.
50. 1972 Standard Specifications for Construction of Highways, Streets and Bridges. Texas Highway Department, pp. 213-214.
51. Schweyer, H. E., and Gartner, W. A Study of Tenacity of Aggregates in Surface Treatments. Highway Research Record 104, 1965, pp. 18-35.
52. Robinson, D. A. Roads Surface Dressing in Sweden. Roads and Road Construction, Vol. 42, Nov. 1964.
53. Swaminathan, C. G., and Shulka, R. S. Fluxing of Bitumen for Surface Dressing. Indian Roads Congress, Road Res. Bull. 9, 1964.
54. Swami, S. A. Australian Method of Surface Dressing. Jour. Institution of Highway Engineers, Vol. 12, Jan. 1965.
55. Coulter, T. M., and Punch, J. J. Design, Construction and Maintenance of Roads and Runways. 11th Permanent Internat. Assoc. Road Congress, Rio de Janeiro, 1959.
56. Shelburne, T. E. Crushing Resistance of Surface-Treatment Aggregates. Purdue University, Engineering Bull., Research Series No. 73, Vol. 23, No. 5, Sept. 1940.
57. Schonfeld, R. Photo-Interpretation of Skid Resistance. Highway Research Record 311, 1970, pp. 11-25.
58. Gallaway, B. M., Schiller, R. E., and Rose, J. G. The Effects of Rainfall Intensity, Pavement Cross Slope, Surface Texture, and Drainage Length on Pavement Water Depths. Texas Highway Dept., Res. Rept. 138-5, July 1971.
59. Tabor, D. Hysteresis Loss in the Friction of Lubricated Rubber. Engineering, Vol. 186, 1958, pp. 838-840.
60. Van der Burgh, A. J. P., and Obertop, D. H. F. Surface Texture and Non-Skid Properties of Road Surfaces. Proc. Internat. Skid Prevention Conf., Virginia Council for Highway Investigation and Research, Part 2, 1959, pp. 489-496.
61. Maclean, D. J., and Shergold, F. A. The Polishing of Roadstones in Relation to Their Selection for Use in Road Surfacings. Proc. 1st Internat. Skid Prevention Conf., Virginia Council for Highway Investigation and Research, Part 2, 1959, pp. 497-508.
62. Shupe, J. W., and Goetz, W. H. A Laboratory Investigation of Pavement Slipperiness. HRB Bull. 219, 1959, pp. 56-73.
63. Whitehurst, E. A., and Goodwin, W. A. Tennessee's Method of Laboratory Evaluation of Potential Pavement Slipperiness. Proc. 1st Internat. Skid Prevention Conf., Virginia Council for Highway Investigation and Research, Part 2, 1959, pp. 329-332.
64. Colley, B. E., Christensen, A. P., and Nowlen, W. J. Factors Affecting Skid Resistance and Safety of Concrete Pavements. HRB Spec. Rept. 101, 1969, pp. 80-99.

EXPERIENCES WITH SKID-RESISTANT EPOXY ASPHALT SURFACES ON CALIFORNIA TOLL BRIDGES

Charles Seim, California Department of Transportation

Epoxy asphalt surfaces have been placed on 3 bridges to solve very special surfacing problems unique to each bridge. The surfaces fall into 3 general categories: pavements thicker than 1½ in. (3.8 cm) applied to orthotropic steel plate decks, pavements from ½ to 1 in. (1.3 to 2.5 cm) applied to old concrete decks that are structurally sound but have deteriorated as a riding surface, and chip seals ¼ in. (0.6 cm) or less in thickness also applied to old concrete decks that are structurally sound but need sealing and improved skid resistance. In addition to high skid resistance, each case has one other controlling factor. For the first case, the factor is flexure fatigue controls; hence, a pavement resistant to fatigue is required. For the latter 2 cases, the factor is superimposed weight on the existing bridge controls; hence, thin but durable surfaces are required. In all 3 cases, epoxy asphalt provided a thoroughly satisfactory solution.

•THE CALIFORNIA Department of Transportation manages 8 of the 9 toll bridges located within the State of California. The newest is the San Diego-Coronado Bay Bridge, opened in 1969. The list also includes 2 world-reknowned bridges: the San Francisco-Oakland Bay Bridge, opened in 1939, and the new San Mateo-Hayward Bridge, opened in 1967. This paper deals with these 3 bridges on which epoxy asphalt surfaces have been placed to solve very special surfacing problems unique to each bridge. These surfaces fall into 3 general categories:

1. Pavements thicker than 1½ in. (3.8 cm applied to orthotropic steel plate decks);
2. Pavements from ½ to 1 in. (1.3 to 2.5 cm) thick applied to old concrete decks that are structurally sound but have deteriorated as a riding surface; and
3. Chip seals ¼ in. (0.6 cm) or less in thickness also applied to old concrete decks that are structurally sound but need sealing and improved skid resistance.

In addition to high skid resistance, each case has one other controlling factor. For the first category, the factor is flexure fatigue controls; hence a pavement resistant to fatigue is required. For the latter 2 cases, the factor is superimposed weight on the existing bridge controls; hence thin but durable surfaces are required. In all 3 cases epoxy asphalt was thoroughly satisfactory.

EPOXY ASPHALT PAVEMENT

Epoxy asphalt pavement is similar to ordinary asphalt concrete pavement except that epoxy resin, hardeners, and other modifiers are added to the asphalt to produce a 2-component binder. It was originally developed for airfields to resist jet fuels and heat blasts (1, 2). It is mixed in the pug mill of a conventional hot plant, delivered in dump trucks, and placed and compacted by conventional equipment. A tack coat of the epoxy binder is applied to firmly bond the pavement to the substrate. After placement and rolling, the pavement has about the same properties as ordinary asphalt concrete; after cooling, it can be opened to traffic. The epoxy requires 30 to 60 days to become fully cured depending on the ambient temperature. When fully cured, epoxy asphalt is

Publication of this paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction.

a thermoset plastic similar in strength to portland cement concrete but with the flexibility of asphalt concrete. Typical properties are given in Table 1 (3).

The conventional batch plant must be modified slightly to heat, meter, and inject the 2-component binder directly into the pug mill. To improve bond, the substrate is sprayed with a tack coat of the same epoxy binder as is used in the mix. Temperature of all ingredients is critical and must be carefully controlled before and after mixing. When the plant is producing epoxy asphalt it cannot be used to produce ordinary asphalt mixes.

The mix material in the truck must be placed within a specified time limit. Holding too long will not allow proper compaction or the mix may set in the paving machine. Placing the material too early will prevent full cure and cause some loss of strength. Rolling must begin immediately with both steel and pneumatic tire rollers. Although epoxy asphalt is more difficult to use than conventional asphalt, a paving contractor can control it after he or she learns the necessary skills.

EPOXY ASPHALT CHIP SEAL

Epoxy asphalt chip seal is similar in form to ordinary asphalt chip seal except that an epoxy-modified asphalt binder is used. Typical properties are shown in Table 1 (4). The concrete deck is cleaned and sprayed with a flood coat of a 2-component epoxy asphalt. Then, stone chips are dropped into the binder. After a 2- to 4-hour cure period (depending on ambient temperatures), the area is swept to remove excess chips that have not bonded and the surface is ready for traffic (5).

The 2-component binder must be accurately heated, metered, mixed, and sprayed on the concrete deck in a uniform application. It is not the same binder as that used in epoxy asphalt pavement. A specially constructed 2-component distributer truck is used to spray the binder on the concrete deck. The aggregate is distributed by a side-dumping spreader. Temperature of the binder is critical, and the aggregate must be applied directly behind the sprayer. Epoxy asphalt chip seal must be applied by a skilled contractor with the proper equipment.

SKID RESISTANCE OF EPOXY ASPHALT SURFACES

The thermosetting properties of epoxy asphalt surfaces provide the basis for high skid resistance if epoxy asphalt is used with a selected, tough, abrasion-resistant, and polish-resistant aggregate. The thermosetting epoxy asphalt binder will not flush to the surface or migrate within the pavement. Heat, oil, or gasoline has little effect on it. The epoxy asphalt binder holds the aggregate in a strong thermosetting grip that is durable under heavy traffic conditions. Traffic wear and ultraviolet chalking remove excess hardened binder from the valleys and crevices thus providing macrotexture and maximum exposure of the microtextured faces of the aggregate (6).

Table 1. Typical properties.

	Epoxy Asphalt				
Properties	Binder for Concrete	Binder for Chip Seal	Concrete	Asphalt Concrete	Portland Cement Concrete
Tensile strength at 73 F, psi	190	2,000	600	130	350
Tensile elongation at break at 73 F, percent	215	50	3	1.2	0.3
Flexural strength at 77 F, psi			640	80	600
Flexural modulus at 77 F, psi × 10^3			380	200	3,000
Marshall stability at 140 F, pounds			14,400	2,685	
Tensile bond strength to steel, psi			220	30	
Tensile bond strength to PCC, psi		400	240	50	
Marshall stability after immersion in JP-4 jet fuel for 24 hours at 140 F, pounds			13,200	Disintegrated	
Stability at 400 F after 2 hours at 400 F, pounds			4,200	Nil	
Hveem stabilometer value at 140 F			73	25	

Note: 1 F = 1.8 (1 C) + 32. 1 psi = 6.89 kPa. 1 pound = 4.4 N. 1 mil = 25.4 μm.

TEST AREAS

Open-Graded Epoxy Asphalt Pavement

The San Francisco-Oakland Bay Bridge (SF-OBB) was modified and strengthened from 1958 to 1963 to carry 5 lanes of automobile and truck traffic on both its upper and lower decks. In 1963 the upper deck was surfaced with an epoxy coal-tar chip seal. The lower deck was surfaced the next year with the same system.

In 1968 specimens taken from the deck showed that particles had been picked out, polished, or sheared off to the level of the resin matrix. This loss of aggregate was attributed to the rounded shape of the grains, low adhesion of the epoxy coal-tar binder to the slick silica surface, and low particle toughness. The deck was structurally sound but had deteriorated as a riding surface. In late 1969, 2 large test areas on horizontal curves were paved with an open-graded epoxy asphalt ½ in. (1.3 cm) thick (7). Two different types of aggregate were used: an air-cooled blast furnace slag and a granite. As expected, wet weather accidents at these 2 locations were reduced dramatically.

Average daily traffic in these test areas is approximately 17,000 vehicles per lane, about 10 percent of which is heavy trucks. Both types of aggregate have raveled a little but are still performing satisfactorily as a pavement.

Dense-Graded Epoxy Asphalt Pavement

In a search to find a surface for the remainder of the SF-OBB, 16 test areas were placed on the upper deck of the bridge in the summer of 1971 to test a variety of pavements and chip seals. Six of these areas are dense-graded epoxy asphalt pavements, 1 is epoxy asphalt chip seal, and the remaining 9 are chip seals using polyester, urethane, or epoxy resins as binders. Each test area is a full lane-width wide and approximately 200 ft long.

For the 6 areas of dense-graded epoxy asphalt, 3 different types of aggregate were used in ½-in. (1.3-cm) and 1-in. (2.5-cm) thicknesses. Surfaces had to be thin to keep down the weight added to the bridge. Test patches 1 and 2 contain an expanded shale, lightweight aggregate conforming to ASTM C 330-69 with the surface sealed by firing after crushing. Test patches 3 and 4 used a blast furnace slag, and test patches 5 and 6 used a granite aggregate. The latter 2 were the same as those used for the open-graded pavement areas placed in 1969. Average daily traffic is the same as that for the open-graded pavement test areas.

The San Mateo-Hayward Bridge (SM-HB), paved with a dense-graded epoxy asphalt in 1967 (8, 9), and the San Diego-Coronado Bay Bridge (SD-CBB), also paved with a dense-graded epoxy asphalt in 1969, provided several older test areas for skid-resistance monitoring. Both bridges have orthotropic steel deck spans for which dense-graded epoxy asphalt pavement is an ideal paving material because of its proven resistance to flexure fatigue (10, 11). Pavement thickness for both bridges varies from 1½ to 2 in. (2.5 to 5.0 cm).

The orthotropic portion of the SM-HB is paved with a calera limestone and is designated test patch G. Average daily traffic is approximately 4,400 vehicles per lane, over 900 of which are heavy trucks.

The orthotropic portion of the SD-CBB is paved with a rhyolite aggregate. The outer lanes are designated as test patch O. The center lane of the 5-lane bridge is a switch lane that is opened to traffic only a few hours a day. This lane is designated as test patch D. Thus, 2 areas of different traffic exposure were available for skid resistance monitoring. Average daily traffic on the outer lanes is approximately 5,900 vehicles per lane, about 3 percent of which is trucks.

Epoxy Asphalt Chip Seal

The epoxy asphalt chip seals placed on the SF-OBB designated as test patch 14 used a calcined bauxite aggregate. This type of aggregate is manufactured from bauxite mined in British Guiana, South America, by calcining at 1 600 C. The resulting particle is about 85 percent aluminum oxide and has a minimum Mohs' scale hardness of 8. Traffic exposure is the same as that for the other test areas placed on the upper deck.

EVALUATION OF TEST SURFACES

A total of 12 test areas that used epoxy asphalt with different types of aggregate and different traffic exposures were available for evaluation. Although over 35 test areas were included in the study, only the test areas containing epoxy asphalt are reported here. The following 6 evaluation factors were used to judge the surfaces:

1. Visual inspection to determine obvious wear, raveling, or structural deterioration;
2. Pull-out force on 1-in. (2.5-cm) and 2-in. (5.0-cm) round cores to determine bond strength;
3. Macrophotographs of the surface to determine wear, polish, and aggregate loss;
4. Polish values of the aggregate as determined by the British wheel test to determine relative polishing values;
5. Los Angeles Rattler values to obtain aggregate toughness; and
6. Towed trailer skid tests to determine skid resistance of the surfaces.

Visual inspection in the fall of 1973 indicated all epoxy asphalt pavements and chip seals were performing satisfactorily. Raveling was continuing at a very slow rate in the open-graded mixes. Some rutting was evident in the dense-graded mixes placed on the concrete deck particularly in the lightweight aggregate test areas. The rutting was believed to be caused by the wheel rims of flat tires. Some aggregate loss was evident on the epoxy asphalt chip seal.

Pull-out values varied quite markedly, ranging from a low of a few pounds per square inch to well over 200 psi (1 378 kPa). The highest values were obtained for pavement placed on steel decks. The low values are associated with the old epoxy coal tar layer that was not removed in all areas because of the additional cost of removal.

Magnified photographs of the surface show some aggregate loss and some points of wear or polish on certain types of aggregates.

Los Angeles Rattler values were obtained from the California Department of Transportation laboratory. All aggregates met the requirements for class A aggregates.

Polish values of various aggregates were determined by the Texas Highway Department, which had one of the few British wheel testers in the United States in 1973. The Texas laboratory used a modified method to determine polish values, so the values reported here may not be directly comparable to values obtained from other laboratories (12). However, the values are useful because they are relative and can be used to rank the aggregates according to their polishing susceptibility. Table 2 summarizes the Los Angeles Rattler values and the polish values for the aggregates used in these epoxy asphalt test areas.

TOWED TRAILER SKID RESISTANCE STUDIES

Despite its limitation the most practical device for monitoring skid resistance of surfaces in the field is the towed trailer tester (ASTM E 274-70). The California Department of Transportation's 2 trailers were used to evaluate more than 35 test areas

Table 2. British wheel polish and Los Angeles Rattler values.

		British Wheel Polish		Los Angeles Rattler	
Aggregate	Classification	Value	Expected Range	100 Revolutions	500 Revolutions
Air-cooled slag	Blast furnace slag, moderately vesicular	38	38 to 44	6	29
Granite	Hornblend granite	39	34 to 40	4	20
Lightweight	Lightweight synthetic expanded shale	47	40 to 53	5	25
Limestone	Dolomite limestone and chert	32	25 to 40	5	23
Rhyolite	Rhyolite/andesite, weathered	42	33 to 42	4	17
Calcined bauxite	Synthetic	44	—	6	27

Table 3. Comparison of composite SNs.

Bridge	Surface	Aggregate	Date Placed	Wear Factor 9/72	Wear Factor 10/73	Composite $SN_{40/55}$ 9/72	Composite $SN_{40/55}$ 10/73
SF-OBB	½ in., open graded	Air-cooled slag	11/69	57	74	(38 + 36) + (34 + 32) = 140	(40 + 34) + (36 + 31) = 141
	½ in., open graded	Granite	11/69	57	74	(44 + 41) + (42 + 39) = 166	(44 + 45) + (43 + 34) = 166
	½ in., dense graded	Lightweight	9/71	19	41	(44 + 34) + (23 + 23) = 124	(47 + 40) + (27 + 22) = 136
	1 in., dense graded	Lightweight	9/71	19	41	(44 + 34) + (29 + 17) = 124	
	½ in., dense graded	Air-cooled slag	9/71	19	41	(42 + 38) + (33 + 30) = 143	(47 + 35) + (29 + 24) = 135
	1 in., dense graded	Air-cooled slag	9/71	19	41	(44 + 39) + (36 + 32) = 151	(45 + 46) + (40 + 38) = 169
	½ in., dense graded	Granite	9/71	19	41	(36 + 28) + (20 + 14) = 98	(34 + 28) + (25 + 17) = 104
	1 in., dense graded	Granite	9/71	19	41	(36 + 28) + (17 + 20) = 101	(43 + 31) + (23 + 18) = 115
	Chip seal	Calcined bauxite	9/71	19	41	(62 + 56) + (51 + 51) = 220	(62 + 58) + (57 + 48) = 225
SM-HB	1½ in., dense graded	Limestone	10/67	25	32	(52 + 48) + (37 + 33) = 170	(56 + 47) + (41 + 23) = 167
SD-CBB	1½ in., dense graded	Rhyolite	7/69		20		(54 + 46) + (42 + 31) = 173
	1½ in., dense graded[a]	Rhyolite	7/69		8		(59 + 56) + (48 + 40) = 203

Note: 1 mile = 1.6 km. 1 in. = 2.5 cm.

[a]Center lane.

included in our studies although only 12 test areas containing epoxy asphalt are reported here.

The drop in skid number (SN) with increasing speed is well known. One measure of a superior surface would be only a slight drop, or, better, a constant SN with increasing speed. This suggests that an SN reading at a higher speed could be added to the standard 40-mph (64-km/h) reading as a means of comparing one surface with another.

A smooth tire braking on a wet surface depends on the pavement macrotexture for the escape of water. Thus, an SN produced by a smooth tire on the towed trailer tester can be used in a general way to compare the macrotexture of surfaces and tendency toward hydroplaning. The surface with the highest smooth-tire SN should have the greatest macrotexture and raise the speed at which hydroplaning will occur.

Similarly, an SN reading for a smooth tire at a higher speed could be added to the aforementioned readings as a further means of comparing surfaces.

If the SNs for 40 and 55 mph (64 and 88 km/h) for ribbed and smooth tires are added together, the resulting composite SN can be used to rate surfaces. The equation used for calculating the composite skid number in this report is: $SN_{40}/_{55} = SN_{40} + SN_{55} + SN_{40S} + SN_{55S}$. The subscript number indicates speed in miles per hour; subscript S denotes a value obtained with a smooth tire.

The high speed value of 55 mph (88 km/h) is about the speed range on most California toll bridges. The 40-mph (64-km/h) value is used because it is currently the standard of comparison. The composite SN can be divided by 4 to lower it to a more familiar range although reporting it as a series of 4 numbers allows a more detailed comparison of surfaces.

Skid numbers were also taken at 20 mph (32 km/h) for both ribbed and smooth tires for the SF-OBB upper deck test areas. This was possible because the area could be coned off from traffic and the slow speed of the towed trailer was not a traffic hazard. Under some traffic conditions, the high speed reading had to be taken at 50 mph (80 km/h).

Table 3 compares the composite SNs for the epoxy asphalt test areas for a 12-month interval. Most of the pavements showed a small gain in the composite SN. Increases in skid resistance are expected during the early life of epoxy asphalt pavements. As excess binder is worn from the faces of particles and removed from crevices, the skid resistance increases to the maximum value possible with the aggregate used.

WEAR FACTOR

The average daily traffic per lane on each bridge varies. The SF-OBB carries approximately 17,000 vehicles per lane (vpl), about 10 percent of which is trucks. The SM-HB is approximately 4,400 vpl, about 20 percent of which is trucks. The SD-CBB count is 5,900 vpl, about 3 percent of which is trucks.

To compare surfaces with different traffic counts, a wear factor was calculated (13). Wear factor is vpl multiplied by pavement age, in years, at the time of testing, divided by 1,000.

DISCUSSION

Graphs of SN as a function of speed for the 12 test patches of epoxy asphalt are shown in Figures 1 to 12. Test patches 1 to 6, 14, and A and B are on the SF-OBB. Test patches A and B (Figs. 1 and 2) are for open-graded pavements with granite and steel slag aggregates. The superior performance of open-graded pavement to dispel water from the tire tracks is indicated by the fact that the smooth tire line is very close to the ribbed tire line and that the speed gradient is flat. Skid resistance is still increasing slightly for both types of aggregates. The composite SN is high.

Test patches 1 and 2 (Figs. 3 and 4) are for a dense-graded pavement with a lightweight aggregate. The speed gradient is quite steep indicating poorer high speed performance. The smooth tire line is well below the ribbed tire line. Both of these observations suggest low macrotexture. Close observation of the pavement and magnified photographs confirm this. This type of pavement apparently tends to wear to a flat plane with little macrotexture. High SNs at slow speeds indicate good microtexture. The composite SN is low.

Test patches 3 and 4 (Figs. 5 and 6) are for a dense-graded pavement with a steel slag aggregate. The speed gradient is flat and the smooth tire line is fairly close to the ribbed tire line. Many other states have had excellent experience with this type of aggregate. The composite SN indicates good overall performance.

Test patches 5 and 6 (Figs. 7 and 8) are for a dense-graded pavement with a granite aggregate. This test section is not performing well. It has the lowest composite SN of all the test sections. The SN at 40 mph (64 km/h) is too low, and the smooth tire speed gradient line is well below the ribbed tire line. This poor performance probably is due to excess binder in the pavement at first placement. The initial skid-resistance values were so low that the surface was sandblasted to remove excess binder. This improved the skid resistance, and improvement should be shown in the future as the binder is removed by wear and oxidation.

Test patch 14 (Fig. 9) is for a chip seal with calcined bauxite synthetic aggregate. The speed gradients are almost ideal—high and very flat—and the smooth tire line is quite close to the ribbed tire. The composite SN is the highest for all the test areas. Calcined bauxite has been used in Great Britain for about 6 years and is just beginning to be used in the United States. It is still quite expensive and can be used only in chip seals. The short life of chip seals compared to that of pavements has been one of their limitations.

Test patch G (Fig. 10) is for a dense-graded pavement with a limestone aggregate for the SM-HB orthotropic steel deck. The speed gradients are fairly flat, although, as expected in a very dense pavement, the smooth tire line is well below the ribbed tire line. The skid resistance still is increasing slightly, which is unusual for limestone aggregate. This pavement is performing very well and showing no signs of distress after nearly 6 years of heavy truck traffic. The composite SN is high.

Test patches O and P (Figs. 11 and 12) are for a dense-graded pavement with a rhyolite aggregate for the SD-CBB orthotropic steel deck. The speed gradients are fairly flat and the smooth tire line is well below the ribbed tire line. The composite SN for the center lane is very high and is high for the other lanes.

SUMMARY

Some of the experiences with skid-resistant epoxy asphalt surfaces on California toll bridges have been presented. A composite SN has been developed that considers the speed gradient between 2 selected speeds for both a ribbed and smooth tire.

Epoxy asphalt with properly selected aggregate can provide satisfactory skid-resistant surfaces to solve special problems and will continue to maintain good skid resistance under heavy traffic.

REFERENCES

1. Tailoring Asphalt for Jet-Age Applications. Chemical Week, Dec. 1959, p.53.
2. Joseph, A. H. Behavior of Epoxy-Asphalt Airfield Pavements, 1963 Inspections. U.S. Army Corps of Engineers, Feb. 1965.

Figure 1. Test patch A, SF-OBB lower deck, open-graded epoxy asphalt concrete with granite aggregate.

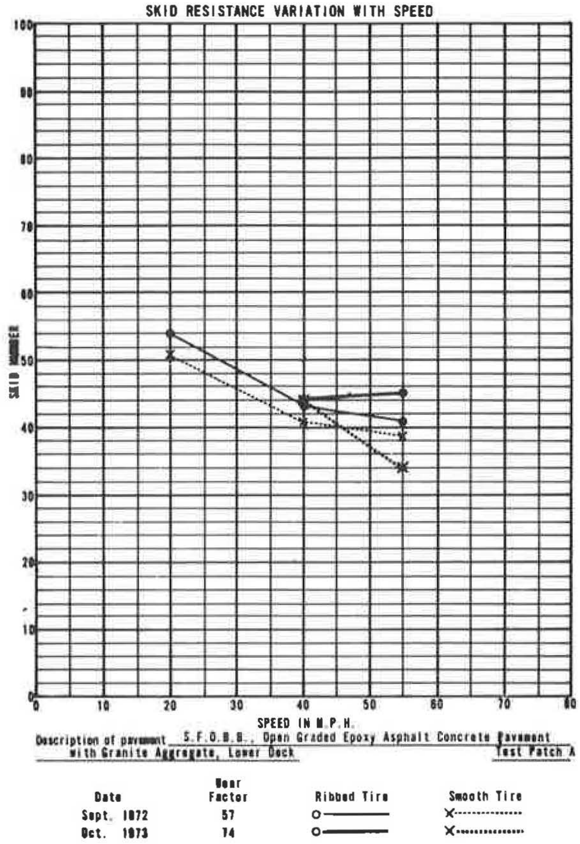


Figure 2. Test patch B, SF-OBB lower deck, open-graded epoxy asphalt concrete with steel slag aggregate.

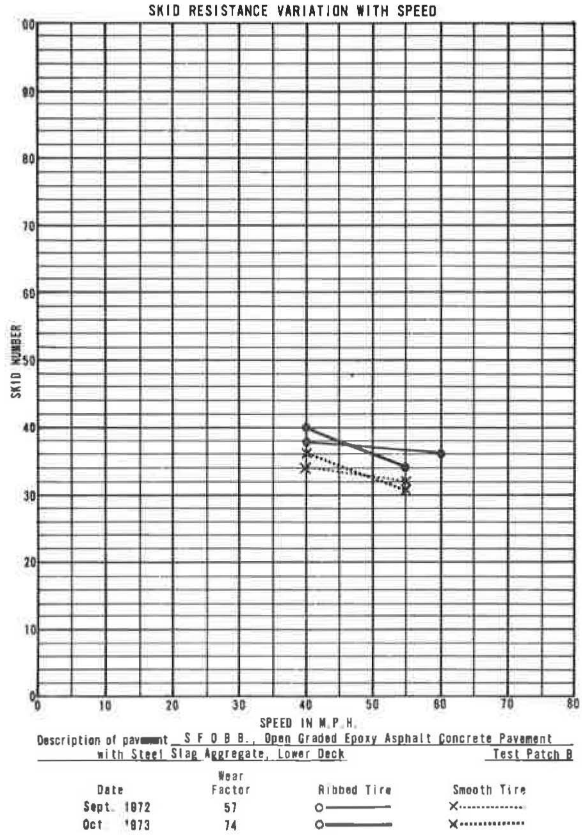


Figure 5. Test patch 3, SF-OBB, 1-in. (2.5-cm) epoxy asphalt concrete with steel slag aggregate.

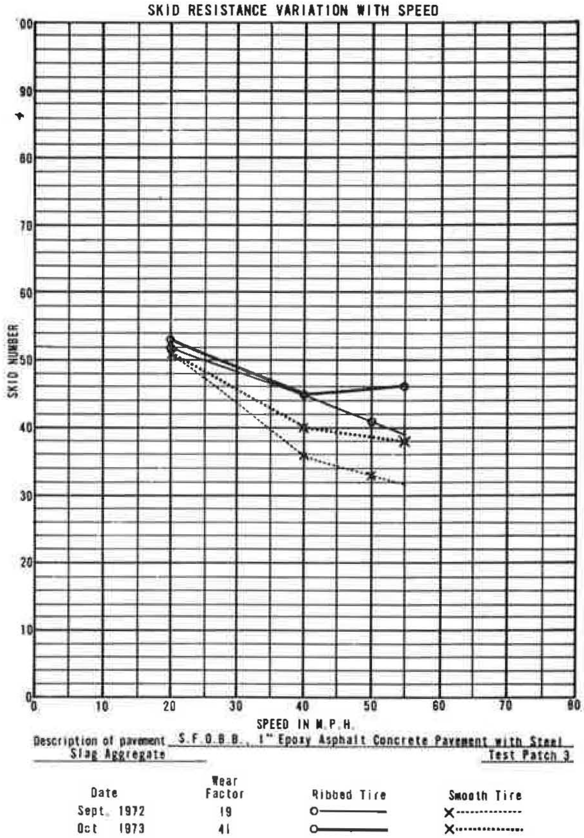


Figure 6. Test patch 4, SF-OBB, ½-in. (1.3-cm) epoxy asphalt concrete with steel slag aggregate.

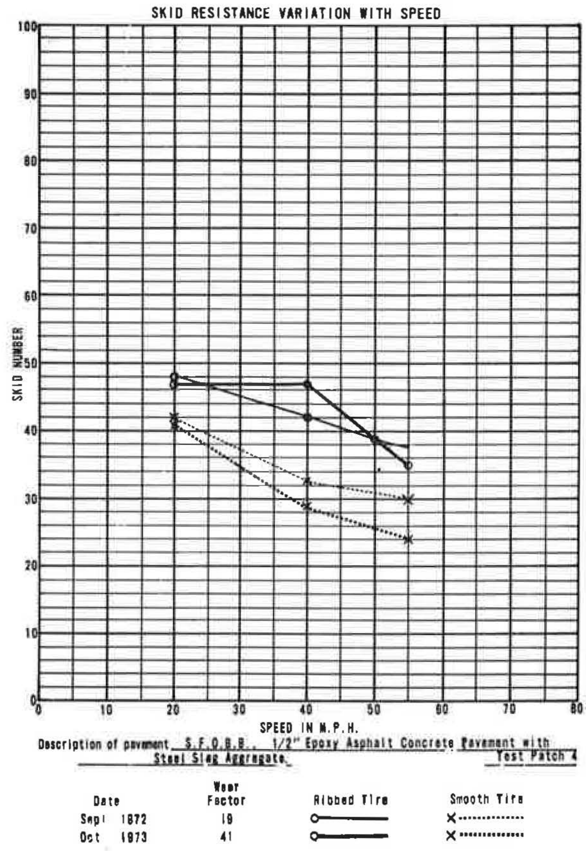


Figure 3. Test patch 1, SF-OBB, ½-in. (1.3-cm) epoxy asphalt concrete with lightweight aggregate.

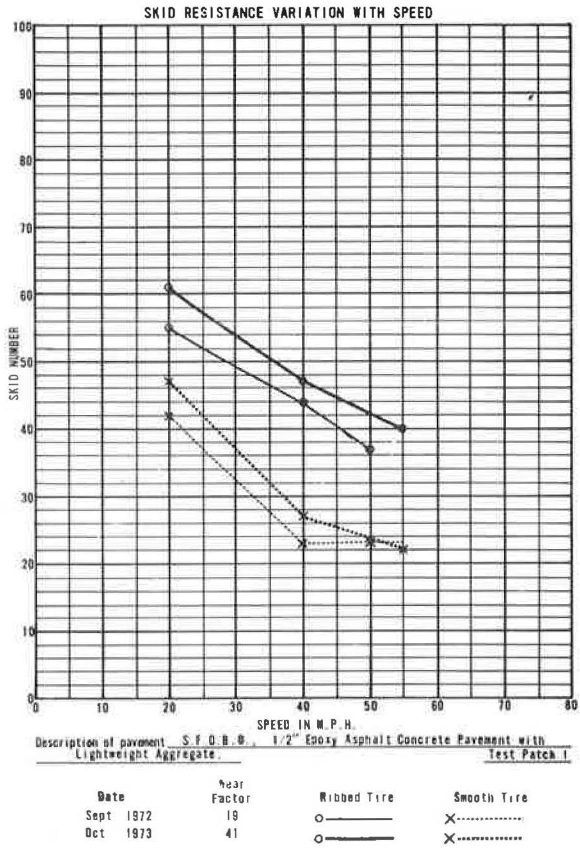


Figure 4. Test patch 2, SF-OBB, 1-in. (2.5-cm) epoxy asphalt concrete with lightweight aggregate.

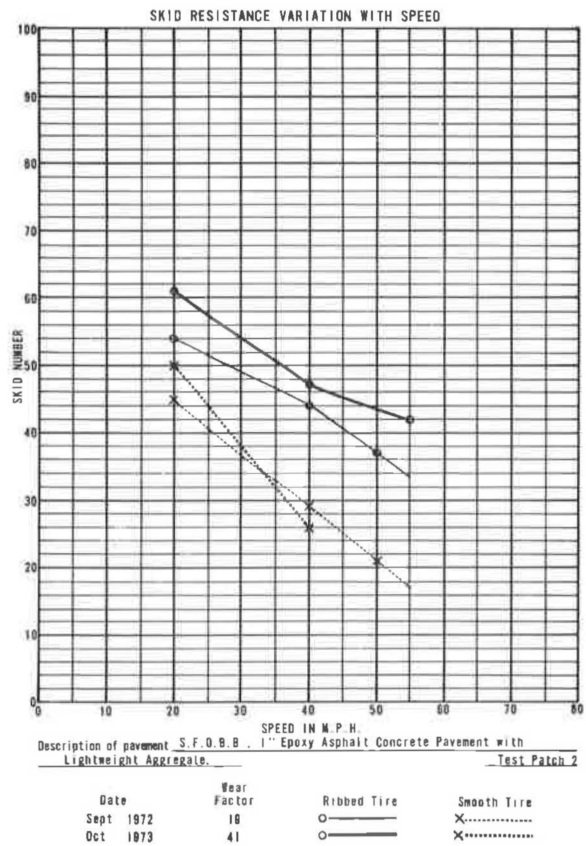


Figure 7. Test patch 5, SF-OBB, ½-in. (1.3-cm) epoxy asphalt concrete with granite aggregate.

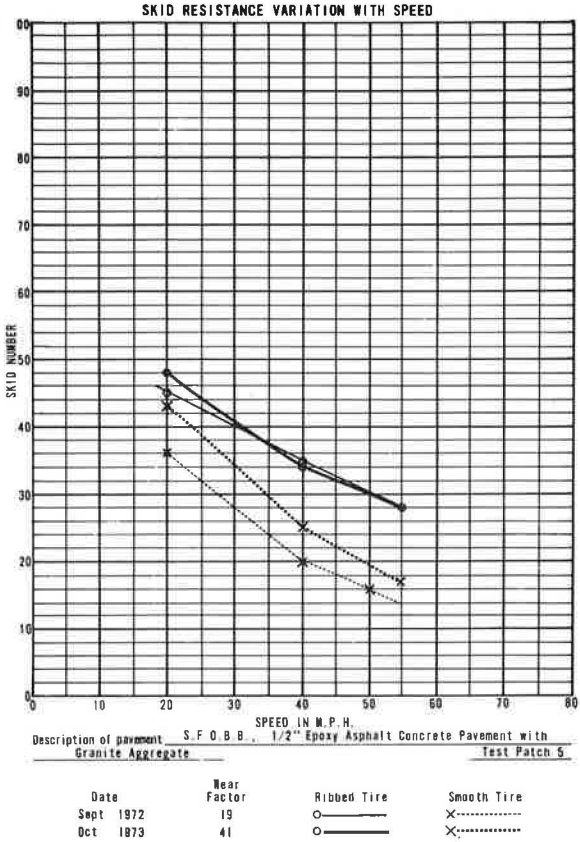


Figure 8. Test patch 6, SF-OBB, 1-in. (2.5-cm) epoxy asphalt concrete with granite aggregate.

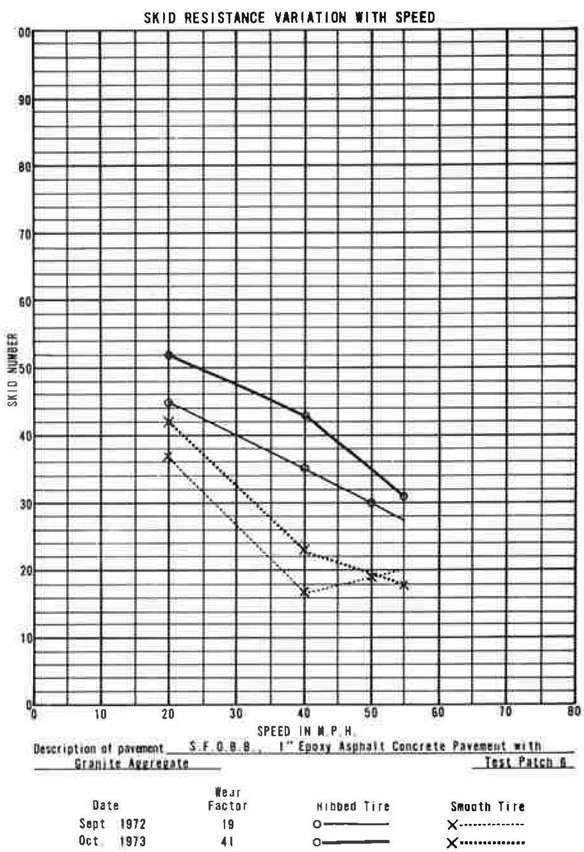


Figure 9. Test patch 14, SF-OBB, epoxy asphalt chip seal with calcined bauxite aggregate.

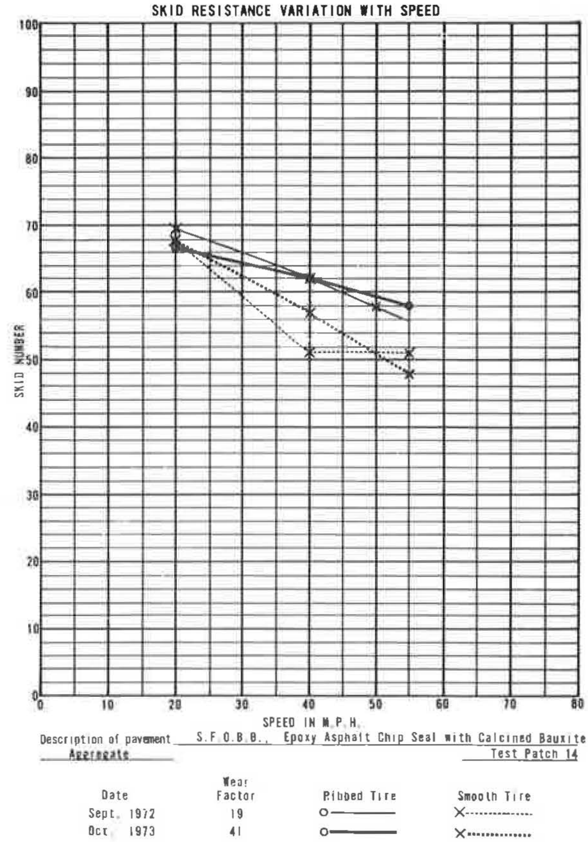


Figure 10. Test patch G, SM-HB, dense-graded epoxy asphalt with calera limestone aggregate on steel bridge.

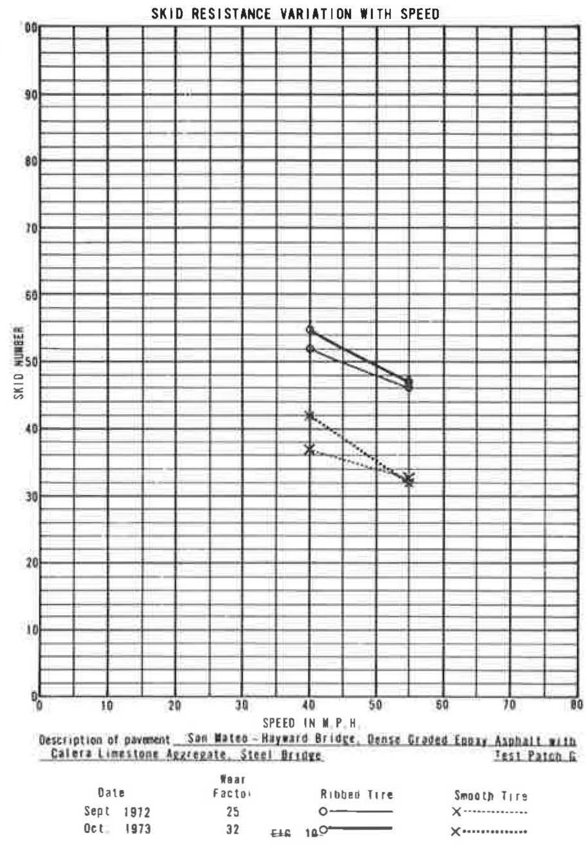


Figure 11. Test patch O, SD-CBB, dense-graded epoxy asphalt with rhyolite aggregate on steel bridge.

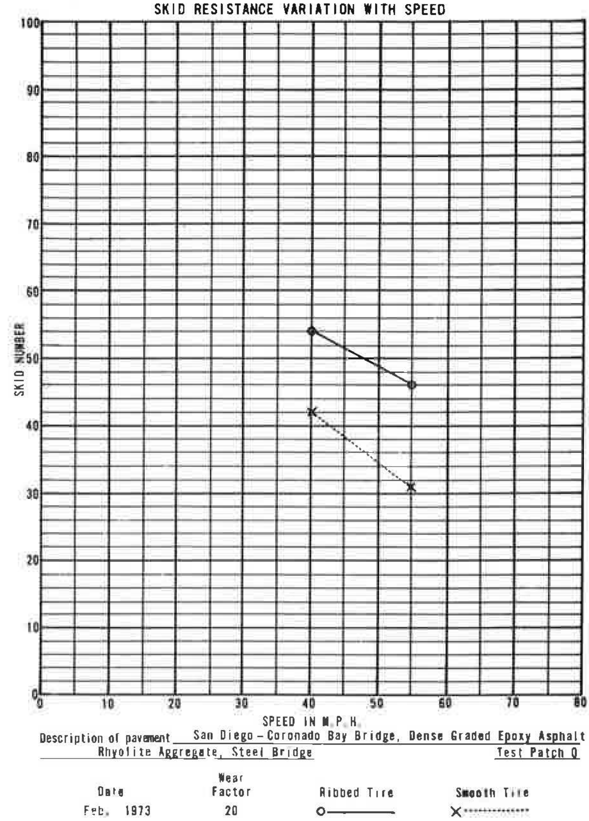
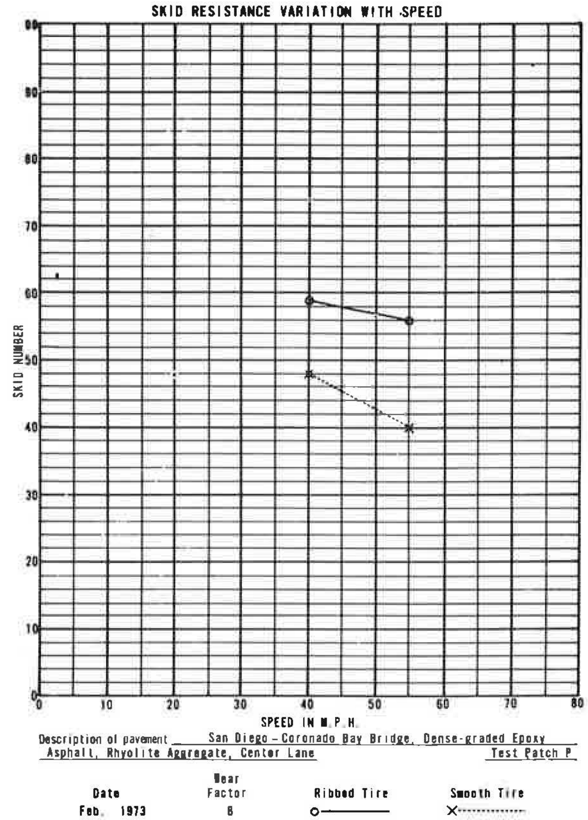


Figure 12. Test patch P, SD-CBB center lane, dense-graded epoxy asphalt with rhyolite aggregate on steel bridge.



3. Specifications for Concresive Epoxy Asphalt Pavement. Adhesive Engineering Co., San Carlos, California.
4. Hatherly, L. W., and Lamb, D. R. Accident Prevention in London by Road Surface Improvements. Shell International Petroleum Co., Ltd., London.
5. Technical Specifications. Adhesive Engineering Co., San Carlos, California.
6. Seim, C. Skid Resistance of Epoxy Asphalt Pavements on California Toll Bridges. ASTM, STP 530, 1973.
7. Brewer, R. A. Epoxy-Asphalt Open-Graded Pavement as a Skid-Resistant Treatment on the San Francisco-Oakland Bay Bridge. HRB Spec. Rept. 116, 1970, pp. 42-47.
8. Balala, B. Studies Leading to Choice of Epoxy Asphalt for Pavement on Steel Orthotropic Bridge Deck of San Mateo-Hayward Bridge. Highway Research Record 287, 1969, pp. 12-18.
9. Balala, B. First Orthotropic Bridge Deck Paved With Epoxy Asphalt. Civil Engineering, April 1968.
10. Metcalf, C. T. Flexural Tests of Paving Materials for Orthotropic Steel Plate Bridges. Highway Research Record 155, 1967, pp. 61-81.
11. Fondriest, F. F. Final Report on Laboratory Evaluation of Various Paving Materials for Orthotropic Steel Deck Bridges. American Iron and Steel Institute, Feb. 1970.
12. Aggregate Polishing Characteristics: The British Wheel Test and the Insoluble Residue Test. Texas Highway Dept., Rept. 126-2.
13. Bransford, T. L. Analysis of Pavement Slipperiness in Alabama. ASTM, STP 530, 1973.

SUMMARY OF FINDINGS OF NCHRP PROJECT 1-12(3), PHASE 1

C. J. Van Til, Materials Research and Development

ABSTRACT

A RESEARCH study (1) is under way to identify and to evaluate systems intended to serve as improved wear- and skid-resistant pavement surfaces. Phase 1 of this study has resulted in the preparation of an extensive annotated bibliography, the development of a method of classification, and the establishment of tentative performance criteria. From a list of all systems that have been used or proposed for use, 10 were selected as suitable for immediate implementation, and detailed recommendations were prepared for the construction of each of them. Thirty-two additional systems considered to be innovative were listed and described. An experimental program to evaluate selected systems is proposed for phase 2 of the study. The proposed program consists of exposing the systems to simulated traffic in a laboratory test track and analyzing the resulting changes in skid resistance, deteriorative wear, and changes in texture. Two sets of test conditions are proposed: one with studded tires, below freezing temperature, wet abrasive, and salt; and the other with unstudded tires, room temperature, wet abrasive, and no salt.

REFERENCE

1. Wear-Resistant and Skid-Resistant Highway Pavement Surfaces. NCHRP Research Results Digest 61, May 1974.

Publication of this abstract sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction.

POROUS SAND-ASPHALT MIXTURES

James H. Havens, Kentucky Department of Transportation

Densely graded sand asphalts are relatively similar to conventional, bituminous concretes. Porous sand asphalts possess the same attributes as other porous, bituminous mixtures. Surely, sand-asphalt mixtures can be designed to be as porous as the so-called open-graded plant-mix seals. Particle shape and texture otherwise define skid resistance. Stability is an ensurance against scaling. Stability is ensured by maximum use of filler-bitumen ratio and the stiffness of asphalt cement.

•OTHERS seem to have forsaken sand asphalts in their search for skid-resistant pavement surfaces. But, the basic principles of the relativity of particle size and packing apply to sand sizes as they apply to pebble sizes. It is still possible to grade sands or silts to obtain a desired porosity. One merely shortens the gradation. Suppose, for instance, that a relatively short-graded sand with sufficient asphalt in it to prevent bulking compacts to 33 percent voids in mineral aggregate (VMA). Suppose, also, that 15 percent voids in the final mixture is desired. For a trial mixture, one might use 13 percent asphalt cement and 5 percent mineral filler (both by volume). For 2.64 specific gravity sand, this would yield about 6.5 percent asphalt by weight. The proportions of asphalt to filler may be adjusted to obtain maximum stability. Stability is necessary to prevent scaling and stripping.

Sands abound in many localities throughout the country. Not all sands are skid resistant. Shape and texture of particles are important attributes. Shape is discernible macroscopically and microscopically. The Goldbeck (1) bulking test is an empirical measure of order, or, conversely, a measure of disorder of shape. Perhaps the most unique feature of sand mixtures, one that has not yet been fully explored and developed, is the strong capillarity and wicking forces that may be achieved and made useful. Whereas drainage of larger, porous mixtures must depend on gravity flow, porous sand mixtures definitely blot and wick and may be designed, hopefully, to siphon.

The objectives are

1. To clear standing water from the pavement as quickly as possible,
2. To minimize hydroplaning, and
3. To avoid polished or polishing aggregate particles that can be lubricated by residual films of water.

Field tests are the final proof of success; however, one should retain an intuitive and scientific skepticism toward standard methods of test such as those now used to measure skid resistance.

RESEARCH AND DEVELOPMENT

From the early 1900s, Kentucky rock asphalt was much admired and respected for its skid resistance qualities. It was basically a porous sand asphalt. Its VMA ranged between 28 and 35 percent. Total voids ranged between 12 and 16 percent. It had an unforgivable weakness—it often scaled or delaminated. The mixture had very low stability when it was fresh. However, a study in 1955 showed an average life expectancy of 17.29 years (2).

Publication of this paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction.

Elsewhere, sand asphalts or sheet asphalts were used as low-cost surfaces in areas in which sands were abundant. Pavement structures consisted of 1 or more courses of sand-asphalt or macadam base and perhaps 2 in. of sand (sheet) surface. Richardson apparently sought angular, sharp, silica sands as early as 1896 (3). Nicholson (4) published photomicrographs of prominent sands in 1926. A dilemma that still persists to some degree was expressed by Gage (5) in 1926, as follows:

> A natural rock asphalt mixture seldom contains much over seven percent of asphalt, yet we are all familiar with what would happen to a sheet asphalt mixture that only contains eight percent of asphalt. I do not think there is much doubt about the durability of some rock asphalt mixtures that do not contain much over seven percent of bitumen yet there is a grave doubt about the durability of the average sheet asphalt pavement that does not contain more than ten percent of bitumen. The stability of a sheet asphalt mixture containing eight and one-half or nine percent of bitumen may be greater than one containing eleven percent, yet the durability of one will certainly exceed that of the other.

Gage (5) apparently was recognizing not only the need for adequate stability but also the prerequisite need for coating thickness. A dense sand asphalt having 10 percent asphalt and a unit weight of 136.67 lb contains 1.58 times more asphalt per cubic foot than an open-graded mixture having 7.5 percent asphalt and a unit weight of 115.68 lb. The percentages by weight of aggregate would be 11.11 and 8.11, respectively. The ratio of the aggregate weights is 1.15:1. Surely, the ratio of the specific surface areas would be greater but not 1.58 times greater. It appears that there has been a tendency in the past to overlook rock asphalts' and other sand mixtures' due portion of asphalt or asphalt and filler. The filler-bitumen ratio appears to have been neglected, and stabilities have not been maximized (6). Scaling and stripping have resulted. Stabilities increase with increasing angularity and texture of the sand, increasing stiffness of asphalt, and increasing the filler-bitumen ratio to optimum. Antistripping agents provide precautionary ensurances against loss of stability in water.

The first generation of sand asphalts in Kentucky were designed to be very dense and stable. They did not scale, but they were not exceptionally skid resistant, and the shape of the sand was not controlled. It seemed necessary to prove that they could be designed to endure and to be as reliable in all respects as high-quality bituminous concrete.

Open-graded sand asphalts compounded to simulate Kentucky rock asphalt were first subjected to road trials in 1968 (7). Sand was selected visually and, after comparison, was judged to be similar but not quite equal to Kentucky rock asphalt sand. Without filler material, the surfaces scaled; with filler material, they did not. About 2 percent filler increased the stabilities from 40 to 80. Marshall stabilities in the range of 500 to 1,200 are thought to be necessary to withstand very high volumes of traffic. A simulated Kentucky rock asphalt has now equaled and slightly exceeded the Kentucky rock asphalt in skid resistance.

A crushed quartz conglomerate sand in a densely graded sand asphalt constructed in 1972 and 1973 compares favorably in skid resistance with a crushed quartz conglomerate in an open-graded plant-mix seal constructed in 1973 on an abutting section of the same road.

It is apparent that the shape and texture qualities sought in sands for sand asphalts are opposite to those sought for portland cement concrete and mortar sands. River sands, glacial sands, especially glacial outwash sands, and beach and blow sands tend to be rounded, frosted, and polished. Many sandstones and conglomerates yield sharp, angular sands. Crushed, manufactured, noncarbonate sands, altogether or blended with qualifying natural sand fractions, may provide new opportunities to use local resources and indigenous material to achieve skid-resistant pavement surfaces. Controlled wear and attrition are essential. The surface, therefore, becomes renewable.

Scaling, shown in Figure 1, is attributed to a deficiency in the design of the mixture—instability. Usually, such mixtures are deficient in asphalt or filler or both. Failures are accompanied by stripping of asphalt from the sand particles. Coatings are thin, and asphalt menisci are not well formed at the interparticle contacts. Persisting water or continual wetness in conjunction with loading accelerates scaling (8). Hard asphalts improve stability, minimize sponginess, and are more resistant to stripping. Anti-stripping agents may be needed with some aggregates. Durable Kentucky rock asphalt surfaces contained asphalt binders (after 15 years in service) having penetrations of about 15. No sand asphalt or rock asphalt having Marshall stabilities of as much as 400 has scaled.

TECHNOLOGY

Sand is defined as aggregate passing the No. 4 or No. 8 sieve. Historically, there have been 2 generic types of sand-asphalt surfacing mixtures: sand asphalts and sand sheet asphalts. Sand asphalts generally have been long-graded sands containing 4 to 14 percent passing the No. 200 sieve and 7 to 11 percent asphalt. Sheet asphalts generally have been finer, containing as much as 98 percent passing the No. 16 sieve, 8 to 16 percent passing the No. 100 sieve, and 7.5 to 12 percent asphalt. The latter is merely a fine sand mixture. A purposeful effort to open-grade and sand mixtures began in 1968 (7).

Aggregate Shape

A cubic packing of uniform spheres, regardless of size, contains 47.64 percent voids (9, 10). Rhombohedral or dense packing contains 25.95 percent voids. Although the isolation of a standard sieve series size of material provides a mixture of sizes in which the smallest particles are half the diameter of the largest, there is a high probability that a random arrangement and distribution of spherical particles in the size range will yield about 41 percent voids. Goldbeck (1) suggested a test in which excess bulking was to be avoided to ensure good concrete-making qualities of fine aggregates. Tests on concrete have indicated an upper limit of about 47 percent. Kentucky has used this type of requirement in its specification for crushed limestone fine aggregate for concrete for several years. Clearly, a size fraction yielding 50 percent or more voids indicates greater disorder in shape, texture, or cohesion.

Test Procedure

Particle shape and texture of each sand shall be controlled so that when it is subjected to the dry bulking test the volume of voids shall be 50 percent or greater. The dry bulking test shall be used as a source control test and thereafter shall be conducted as often as deemed necessary by the engineer. The following describes the test apparatus (Fig. 2).

1. The balance shall have a capacity of 1 500 g and a sensitivity of 0.1 g.
2. The pans for drying samples shall have at least a 1 500-g capacity.
3. The rigid, cylindrical cup shall have an inside diameter of $2\frac{7}{8}$ in. and a height of $5\frac{1}{2}$ in.
4. There shall be a truncated, hollow, metal cone having an overall height of 4 in. and an inside diameter of $5\frac{1}{2}$ in. for the large opening and 1 in. for the small opening.
5. No. 4, No. 8, No. 16, No. 30, and No. 50 size sieves are required.
6. A typical steel straightedge 1 in. by 6 in. by $\frac{1}{16}$ in. is required.

The sample of aggregate shall be washed thoroughly, dried to constant weight at 105 to 110 C, and separated into the following 4 sizes:

Passing	Retained
No. 4	No. 8
No. 8	No. 16
No. 16	No. 30
No. 30	No. 50

Approximately 1 500 g of each of the above sizes shall be required for the tests.

The test shall be conducted on only the specified size fractions having 5 percent or

Figure 1. Early scaling of sand asphalt having low stability and insufficient asphalt.

Figure 2. Test apparatus.

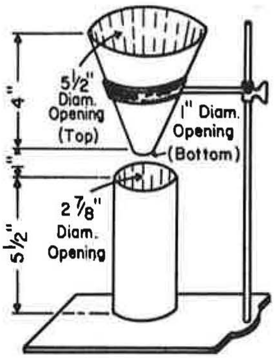


Figure 3. Bulking test for control of aggregate shape.

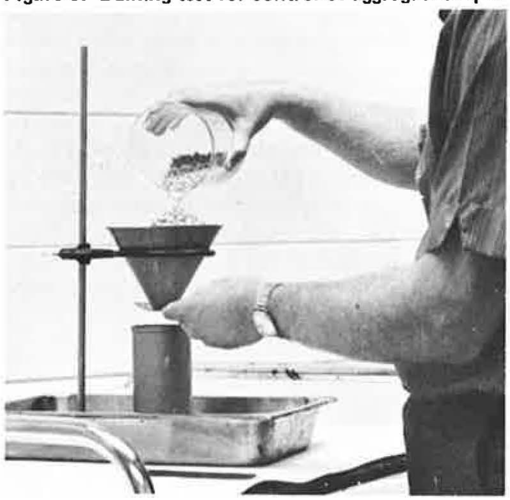
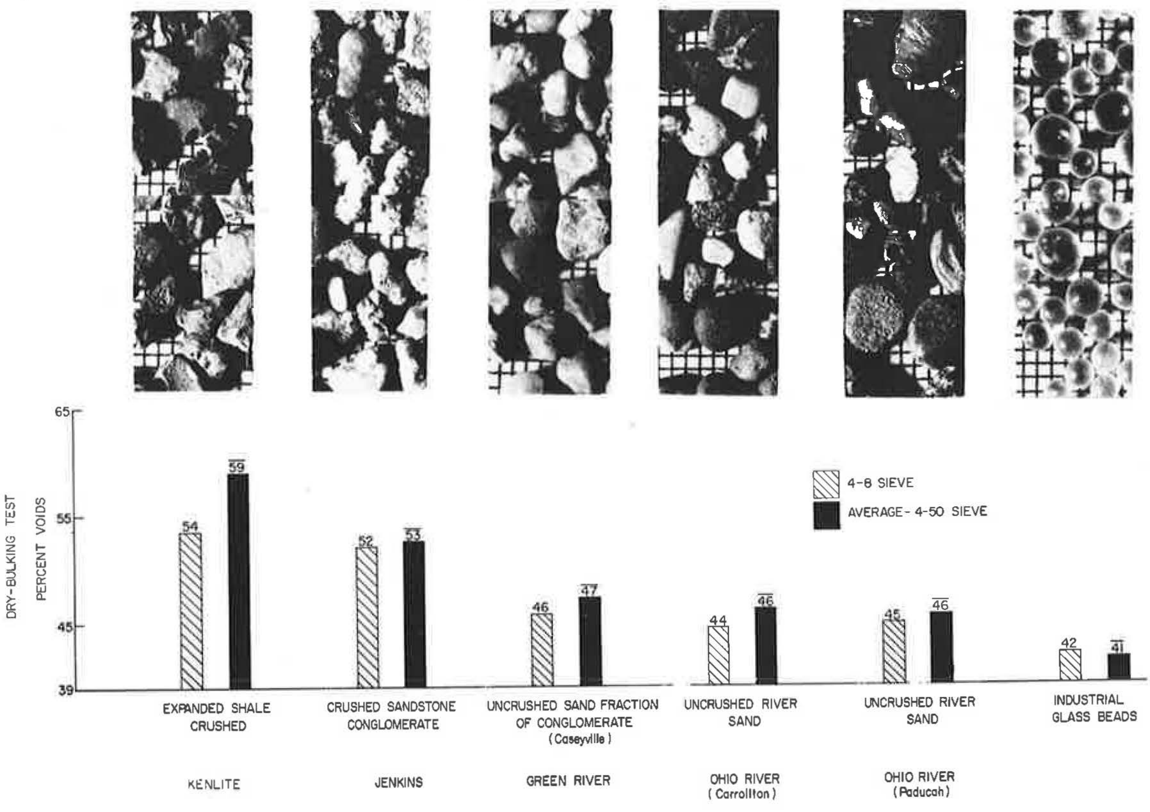


Figure 4. Bulking value of sands associated visually with particle shape, No. 4 sieve size.



more of the aggregate by weight contained in the given specified size fraction.

A size of the aggregate shall be poured into the funnel while a stiff piece of metal is held against the bottom opening. The funnel shall be filled until the material is heaped between 1 and 2 in. above its top level; care shall be taken not to overflow the funnel or to spill material into the cylinder below. The piece of metal used to close the bottom opening of the funnel shall be quickly withdrawn in a horizontal movement and the material permitted to flow freely into the cylinder until it overflows. Then, the flow of the material onto the filled cylinder shall be cut off, and any of the material remaining in the funnel shall be allowed to flow into a pan.

The material in the cylinder then shall be carefully struck off even with the top of the cylinder with the straightedge. This is accomplished by holding the straightedge in both hands, edge down; starting at one side, strike off the material above the top of the cylinder. The straightedge is then placed along a diameter of the cylinder and the material struck off again. This is then repeated in the opposite direction. Extreme care shall be taken during the striking-off operation to avoid any downward pressure on the aggregate or any jarring of the cylinder.

After carefully removing any material that may be adhering to the outside of the cylinder, the weight of the contents shall be determined to the nearest 0.1 g.

The aggregate in the cylinder then shall be recombined with the excess of the same size from the pan, thoroughly mixed, and 2 additional determinations made. An average of 3 determinations having a maximum variation of 4 g shall constitute a test.

The percent of voids in each size shall be determined by the following formula:

$$\text{Percent voids} = 100\left(1 - \frac{W}{VG}\right)$$

where

W = average weight of material in the cylinder,
V = volume of cylinder in cubic centimeters, and
G = bulk specific gravity (oven dry) of the aggregate as determined by the applicable portions of ASTM C 127.

The arithmetical average of the voids percentages so determined for the tested size, the sum of the percentages divided by the number of sizes tested, shall be reported. Figure 3 shows the test being performed.

Figures 4 through 7 illustrate, by association, the relationship between the bulking value and particle shape (11). The test is subject to some error when the aggregate is highly vesicular. Errors arise from inherent inability to determine the true, oven-dry, bulk specific gravity.

The bulk specific gravity of slag larger than the No. 4 sieve may be about 2.30; that of fine slag sand passing the No. 30 or No. 50 sieve may exceed 3.00. Expanded shale fines may range from 2.59 to 2.84. Quartz grains approach 2.64. All approach zero percent absorption as the size diminishes. The specific gravity of powdered material is equivalent to that of a voidless mass.

Porosity and Capillarity

A relatively dense sand-asphalt gradation divided at the No. 30 sieve will provide a coarse and a fine sand, each of which will yield about twice the VMA in asphalt mixtures as the original sand. The coarser fraction would not contain filler; it would have to be added. Such sorting might be accomplished in a wet classifier. All filler could be removed and proportioned back into each of the sands at the hot-mix plant. The 2 mixtures would be very much alike in terms of total voids (porosity) but would have very different pore sizes. Capillary rise and wicking would be greater in the finer mixture if the voids remained open and if the internal surfaces were wettable. Both the coarse and fine mixtures would exhibit wicking capabilities. Wicking is merely capillary forces at work in a porous medium.

Capillary rise, siphoning, and wicking are illustrated in Figure 8. Height, H, must

Figure 5. Bulking value of sands associated visually with particle shape, No. 8 to No. 16 sieve sizes.

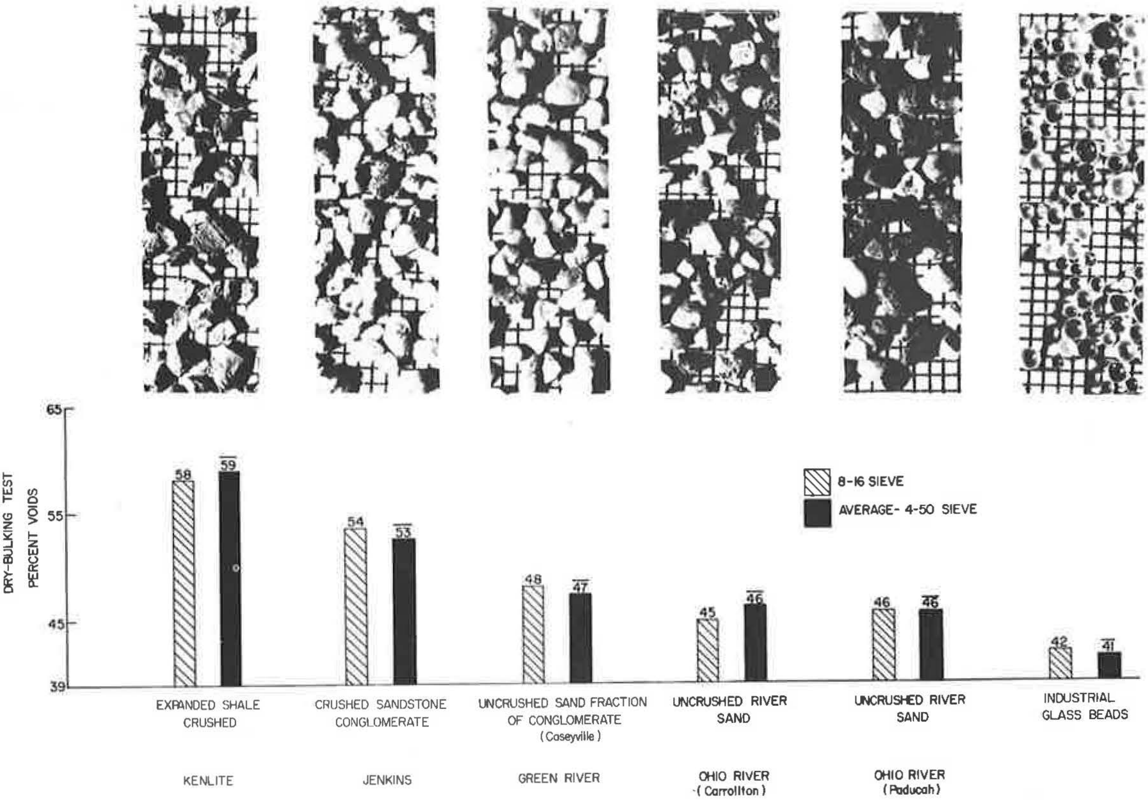


Figure 6. Bulking value of sands associated visually with particle shape, No. 16 to No. 30 sieve sizes.

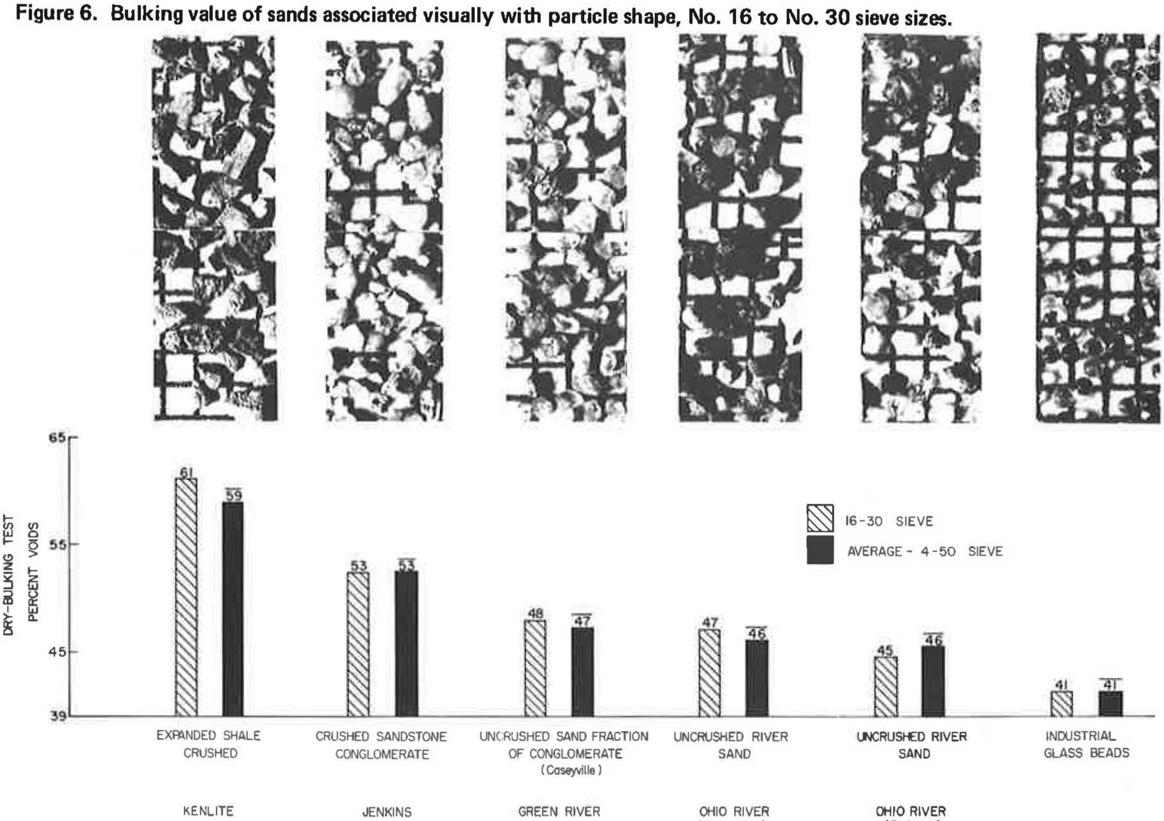


Figure 7. Bulking value of sands associated visually with particle shape, No. 30 to No. 50 sieve sizes.

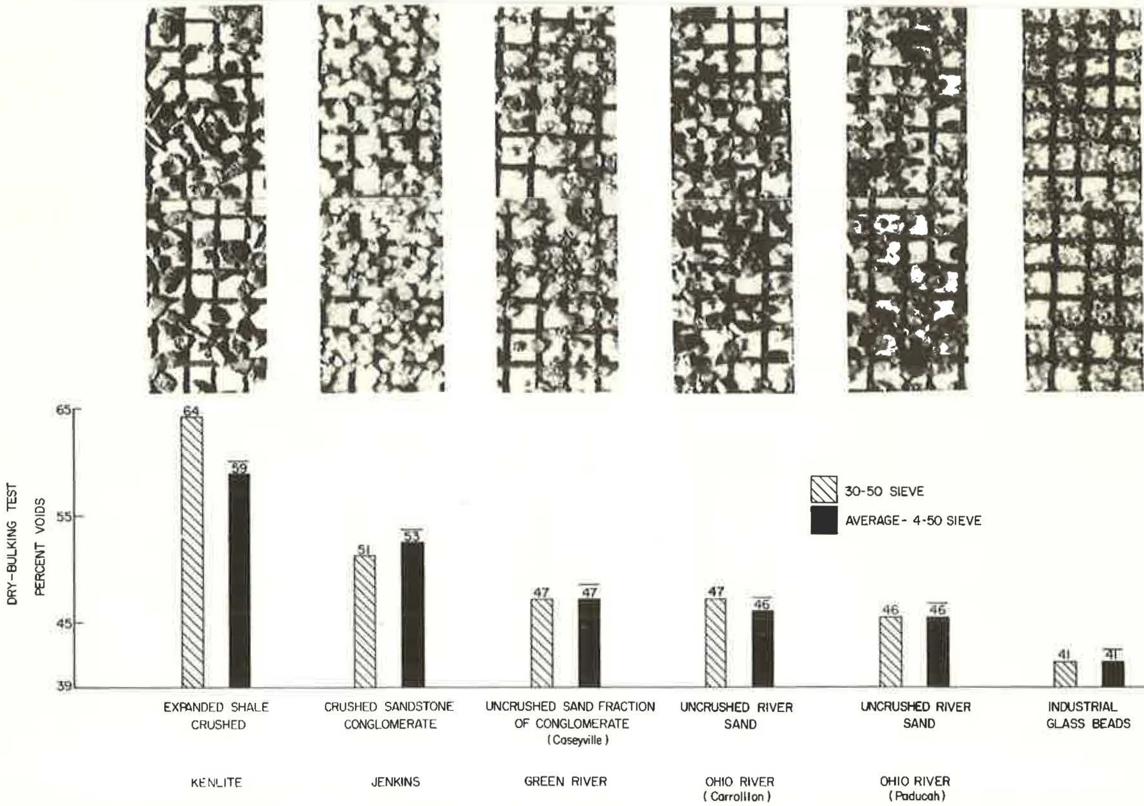
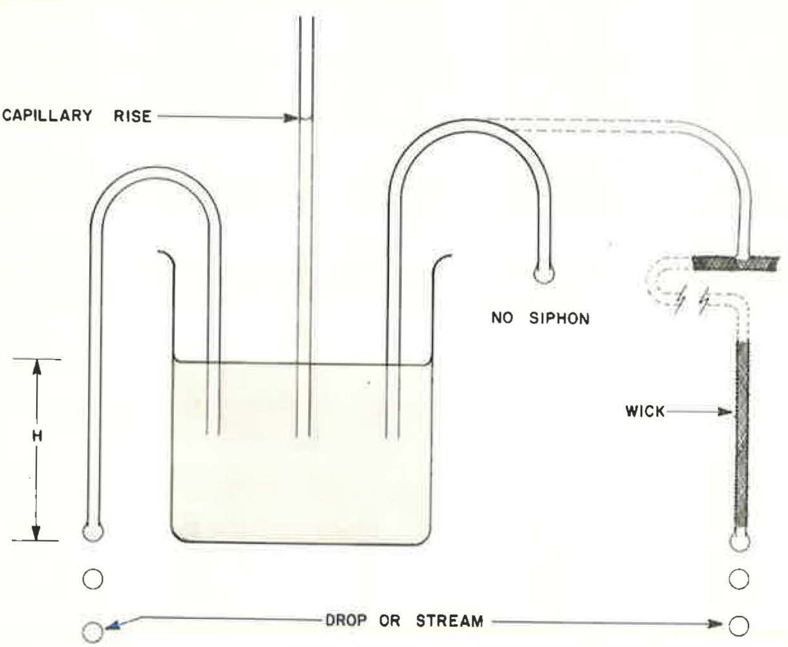


Figure 8. Capillary rise, siphoning, and wicking.



be sufficient to overcome the tensions holding the drop at the tip of the tube before efflux will occur. A wick may substitute for or extend a capillary tube, as shown.

Suppose the capillary rise is 1 cm in a surface course that is 2 cm thick; if the surface course were inundated by rain surface flow and some internal flow would carry the excess water to a lower elevation, the free-water surface would subside into the surface course; ordinary gravity drainage would cease at some point; and shallow basins would siphon (by capillary action) into lower basins and emerge laterally or surface at low points. Water that could form concave surfaces (menisci), because it has shorter radii than the pores causing capillary rise, would not drain; the hydrostatic law does not apply. As drying progresses the menisci recede, but blotting capacity remains high; additional water cast onto the surface is absorbed readily.

Clogging of Pores

Soil and road scum intrusions affect the porosity and surface texture of various types of bituminous surfaces. Dust tends to adhere to fresh asphalt until the asphalt loses its tack. Interior surfaces also retain dust; this mineralizing process improves wettability with respect to water and gives rise to capillary action. Permanent clogging may occur, but the pumping action induced by passing tires during rainy periods tends to flush and clear pores in the wheel paths.

Figure 9 shows a dense sand-asphalt surface outside the wheel paths that is clogged and not readily wettable. The view on the right shows the same drop of water after the addition of a wetting agent. The surface wetted, but there was no in-rush or blotting. In contrast, Figure 10 shows a drying porous sand asphalt (Kentucky rock asphalt) that has been fully wetted and flushed clean in the wheel paths. Blotting occurs readily. Traffic assists the wetting process if the surface is somewhat hydrophobic at first; it also hastens desaturation.

Tack Coat

Porous sand asphalts have a high blotting capacity toward asphalt used in the tack coat. An abundant tack application seems necessary to prevent delamination and to seal the underlying pavement. It is usually prudent to subtract an equivalent amount of asphalt from design asphalt content of the mixture. This adjustment becomes very significant when the sand surface course is very thin.

Particle Orientation and Surface Texture

Particles having 1 or more flat sides tend to be positioned in the surface during compaction so that they present a flat side rather than an acute angle or cutting edge toward the tire. In this position, the edges, if they remain sharp, contribute to tractive resistance. The microtexture of the flat surface may be lost through wear and polishing. Certainly, microtexture is obscured by asphalt when the surface course is new. Skid resistance should improve rapidly for a brief time and then diminish gradually to a more or less constant value. Attrition or loss of particles from an open-graded, plant-mix seal would give an impression of raveling; attrition from a sand surface would be desirable if it occurs uniformly and at a rate commensurate with the life expectancy of surface course. Indeed, a steady wearing away ensures against eventual polishing of sand grains at the surface. Finer sands appear to be more favorable from this point of view than coarse sands. Admittance of finer sands also permits a higher size-reduction ratio and provides greater angularity when sharp sands are manufactured by crushing coarse sands or pea gravel. No. 4 to No. 8 sizes do not appear to be essential to the performance of sand mixtures.

Orientation of particles of fine, crushed quartz sand that are oiled and compressed lightly against a flat surface is shown in Figure 11.

SUMMARY

Skid Resistance

Dense sand asphalts containing blends of crushed limestone sands and natural sands

Figure 9. Quarter-inch-diameter bead of water (left) on dense, clogged sand-asphalt surface not readily wet; wetting agent induces wetting (right) without blotting.

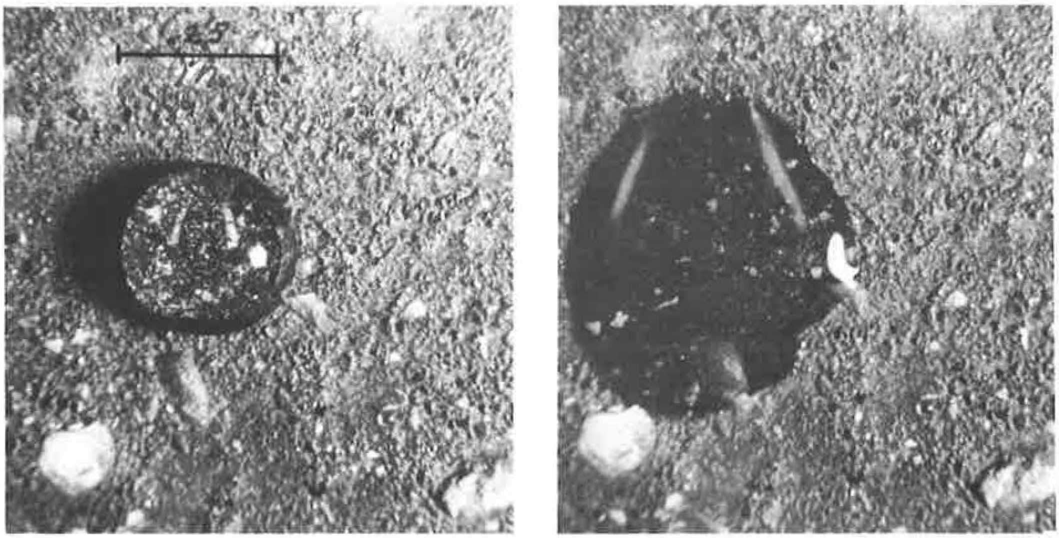
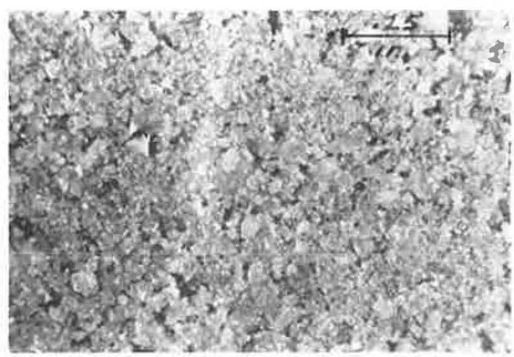


Figure 10. Porous sand asphalt (Kentucky rock asphalt) after rain.

Figure 11. Particle orientation of compressed fine sand; flat sides tend to be horizontal.



have performed about equivalent to bituminous concretes containing natural sands and crushed limestone coarse aggregates. All-limestone sand asphalts have performed about the same as all-limestone bituminous concretes. Sand asphalts containing all-natural sands have tended to exhibit higher skid resistance. Variability in shape, texture, and composition has affected performance. A crushed quartz sand in a dense sand asphalt is showing very good skid resistance after a year under severe traffic. A porous sand asphalt containing selected quartz sand less angular than crushed sand has shown superiority over the same sands in denser mixtures. The ultimate combination of high porosity and sharp, angular (crushed quartz) sands or other hard vesicular sands such as slags, scoria, or expanded shales has not been field tested. Performance equations indicating statistical confidence limits with respect to time and accumulated traffic are yet to be developed. Performance equations are needed to determine which types of surfaces and materials meet minimum standards for skid resistance.

Porous sand asphalts may be considered to possess almost all of the attributes of a popcorn mix or an open-graded plant-mix seal. They do not have comparable pore sizes. For instance, unless a porous sand-asphalt surface is prewet before the ASTM E 274 skid test, the water sprayed in front of the test wheel will not have significant time to wet or be blotted into the surface. The effect on the skid number might be about the same as if the test were made on a dense, nonwet surface. On the other hand, a fully wet, but unsaturated, porous sand asphalt may imbibe the sprayed water very quickly.

Other Attributes

Sand-asphalt surfaces, especially the more porous ones, generate minimum tire noise and tend to damp other noises generated by a vehicle. Under-car noise caused by splash and spray in wet weather is reduced. Significantly, the spray generated by vehicles is reduced unless water is in ponds or the surface is flooded.

REFERENCES

1. Goldbeck, A. T. Stone Sand for Use as Fine Aggregate. Concrete, Vol. 59, 1951.
2. Carter, J. L., Jr. A Study of the Service Life of Pavements in Kentucky. Univ. of Kentucky, MS thesis, 1955.
3. Crane, C. W. Sand for Sheet Asphalt Pavements. Proc. Fifth Annual Asphalt Paving Conf., Asphalt Assn., 1926.
4. Nicholson, V. A Study of the Effect of Shape of Mineral Aggregate on Stability of Asphalt Paving Mixtures. Proc. Fifth Annual Asphalt Paving Conf., Asphalt Assn., 1926.
5. Gage, R. B. Discussion of paper by C. W. Crane. Proc. Fifth Annual Asphalt Paving Conf., Asphalt Assn., 1926.
6. Nijboer, L. W. Plasticity as a Factor in the Design of Dense Bituminous Road Carpets. Elsevier, 1948.
7. Havens, J. H. Hot-Mix Sand Surface. Proc. Kentucky Highway Conf., Univ. of Kentucky, College of Engineering, Bull. 90, Oct. 1969.
8. Majidzadeh, K., and Stander, R. R., Jr. Effect of Water on Behavior of Sand-Asphalt Mixtures Under Repeated Loading. Highway Research Record 273, 1969, pp. 99-109.
9. Farouki, O. T., and Winterkorn, H. F. Mechanical Properties of Granular Systems. Highway Research Record 52, 1964, pp. 10-58.
10. Armstrong, J. C., and Dunlap, W. A. The Use of Particulate Mechanics in the Simulation of Stress-Strain Characteristics of Granular Materials. Texas Transportation Institute, Texas A&M Univ., Res. Rept. 99-1, Aug. 1966.
11. Havens, J. H. Aggregate Shape and Skid Resistance. Proc. Conf. on Skid Resistant Surface Courses, Region 15, U.S. Dept. of Transportation, Chicago Heights, Ill., Sept. 14-16, 1971.

SKID RESISTANCE OF DENSE-GRADED ASPHALT CONCRETE

David C. Mahone, C. S. Hughes, and G. W. Maupin,
Virginia Highway and Transportation Research Council

Descriptions and skid test results are given for Virginia's sand mix, plant-mix seal, dense-graded mix, and urban mix. Although sand mix has provided good skid resistance under most circumstances, problems are encountered in areas of water buildup. Durability problems also are encountered under high-speed truck traffic. Plant-mix seal is extremely useful in situations where water buildup is a problem, but it has possible durability and service-life disadvantages. Experience has shown that dense-graded mixes are durable, and skid test data indicate that good skid resistance is provided at high speeds under heavy water conditions. The urban mix described (a special, dense-graded mix) contains coarse, non-polishing aggregate and fine, polishing aggregate. It has good durability and skid resistance. Of the mixes discussed, the dense-graded mix is the best for skid resistance and economy.

•IN VIRGINIA, the basic concept that has evolved regarding pavement skid resistance is that the level provided should be the optimum. Too little skid resistance obviously would be a hazard to highway safety; too much would be uneconomical and possibly cause neglect of some other feature of the road, such as structural soundness, sub-base drainage, or durability. Working under this concept, Virginia has used several different types of bituminous mixes and has found the dense-graded ones to be the best suited for providing optimum skid resistance on most high-class roads. Table 1 gives the gradations for many of the mixes used in Virginia; the most popular of the dense-graded mixes is S-5.

The skid data shown in this paper were taken by ASTM standard method and by ASTM method modified to include bald tires. The bald-tire tests were run because it is believed that when the water film is as thick as tread depth is deep, a bald-tire situation prevails.

This paper discusses the relationship of tire tread depth to pavement skid resistance and presents skid test data for a sand mix, a plant-mix seal, and several dense-graded mixes.

RELATIONSHIP OF TIRE TREAD DEPTH TO SKID RESISTANCE

Figure 1 shows skid numbers measured at 70 mph (112.7 km/h) by the ASTM trailer method with twice the normal water film [0.040 in. (1.016 mm)] for tread depths of new, $^9/_{32}$-in. (7.1-mm), $^7/_{32}$-in. (5.6-mm), $^5/_{32}$-in. (3.9-mm), $^3/_{32}$-in. (2.4-mm), and bald tires. The tests were performed on a smooth portland cement concrete pavement and on a very harsh one. From the test data it can be seen that, even when twice the amount of water prescribed by ASTM is used, skid resistance is not dependent on tread depth with this type of test method, even on a smooth concrete, until there is less tread than $^3/_{32}$ in. (2.4 mm). It is believed that the 0.040-in. (1.016-mm) water thickness is very

Publication of this paper sponsored by Committee on Characteristics of Aggregates and Fillers for Bituminous Construction.

Table 1. Virginia design specification for bituminous concrete mixtures.

Type	Percent Passing by Weight 1½ In.	1 In.	¾ In.	½ In.	⅜ In.	No. 4	No. 8	No. 30	No. 50	No. 200	Bituminous Materials (percent)	Mix Temperature at Plant (deg F)
S-1						100	95 to 100	50 to 95	25 to 65	0 to 8	8.5 to 10.5	225 to 300
S-2					100	95 to 100	60 to 85	20 to 40	10 to 30	2 to 10	9.5 to 12.0	225 to 300
S-3					100	90 to 100	70 to 95	25 to 55	15 to 35	2 to 12	6.5 to 10.5	200 to 240
S-4				100	90 to 100		60 to 80	25 to 45	10 to 30	2 to 10	5.5 to 9.5	225 to 300
S-5				100	80 to 100		35 to 55	15 to 30	7 to 22	2 to 10	5.0 to 8.5	225 to 300
I-1		100	90 to 100		85 to 100	75 to 100	60 to 95	25 to 60	12 to 35	2 to 12	5.0 to 7.5	225 to 300
I-2		100	95 to 100		60 to 80	40 to 60	25 to 45		5 to 14	1 to 7	4.5 to 8.0	225 to 300
B-1		100	90 to 100			70 to 100	55 to 95	25 to 65	12 to 40	0 to 10	3.0 to 6.5	225 to 300
B-2	100		50 to 75			20 to 35	15 to 25			0 to 5	4.0 to 6.0	200 to 240
B-3	100		72 to 87			35 to 50	28 to 38			2 to 6	4.0 to 7.0	225 to 300
C-1				100	90 to 100	65 to 80	45 to 65	25 to 40	13 to 23	6 to 10	6.0 to 9.0	305 to 345
S-6	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	7.5 to 11.5	225 to 300

Note: 1 in. = 25.4 mm. 1 F = 1.8 (1 C) + 32.

[a]Gradation determined by Marshall design.

Figure 1. Relation of tire tread depth to skid numbers.

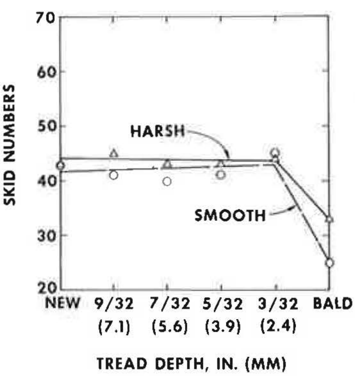
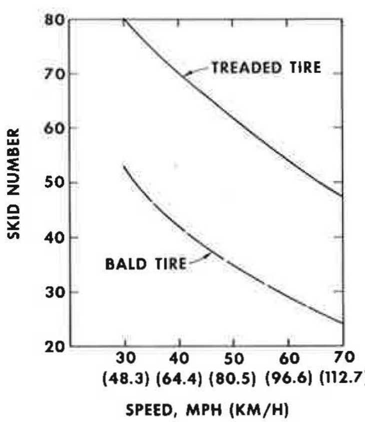


Figure 2. Delamination of sand-mix surface.

Figure 3. Surface texture, sand mix.

Figure 4. Skid test data, sand mix.



similar to water depth during a heavy rain, yet it is realized that at times water films much thicker are experienced.

From a consideration of the curves in Figure 1, one could question whether it is desirable or economically feasible for highway departments to design the majority of pavements for bald-tire skid resistance. We feel that to design all pavements for bald tires would not provide optimum skid resistance and might sacrifice some other feature of the roadway. Of course, where water films are a problem, means for the escape of water must be provided within the pavement surface.

SKID RESISTANCE OF VIRGINIA PAVEMENTS

Sand Mix

Virginia's sand mix is very similar to Kentucky rock asphalt and is used for deslicking pavements (1). It was developed during the extensive deslicking program undertaken in the state several years ago. It served its purpose well and probably still would be in widespread use except for its susceptibility to delamination under heavy, high-speed truck traffic (Fig. 2).

Figure 3 shows a photograph of the wheel path of the sand mix that was skid tested for this study. Figure 4 shows a plot of the test results. The curves in Figure 4 depict the relationship between skid numbers and speed for both the treaded and bald tires. Note that with the standard treaded tires this surface provides excellent skid resistance at all speeds tested. The low values measured with the bald tires indicate that skid resistance is highly dependent on tread depth; thus, the mix should not be used in places where water runoff is poor or where there might be a water buildup from some other cause.

Plant-Mix Seal (Popcorn Mix)

Virginia does not have much experience with open-graded or popcorn mix. The first was placed in 1972, and at present there are only about 12 installations. Two problems already have been encountered—bleeding, shown in Figure 5, and aggregate loss in the wheel path, shown in Figure 6. The latter distress is believed to be due either to an inadequate amount of asphalt in the mix or to some type of stripping.

Figure 7 is a photograph of the surface of a popcorn mix. Figure 8, a graph of the skid test data, shows that, unlike the data for the sand mix, skid numbers for bald tires differ very little from those for the treaded tires; in fact, there probably are no statistically significant differences. Skid numbers for the treaded tire for speeds below 60 mph (96.6 km/h) are not as high as for the sand mix. The fact that the friction-speed curve for this mix is flat, even with bald tires, points out the usefulness of the mix for those places where a water buildup is expected.

Dense-Graded Mixes

To provide durability, strength, and overall good performance, Virginia has relied heavily on dense-graded mixes. This type of mix has a low air void content that prevents the intrusion of air and water, both of which accelerate the deterioration of asphalt. Low air void content is also essential to reduce consolidation under traffic and to provide strength and a long fatigue life. The minimum required density is 92 percent maximum theoretical density (MTD). Proper gradation and asphalt content are essential.

About 90 percent of the surface mixes used in Virginia are densely graded, and they have performed highly satisfactorily. Failures have been experienced, but fortunately the 2 main causes for them have been delineated clearly. One is improper design resulting in overdensification, and the other is the use of polish-susceptible aggregate. The former is inexcusable and the latter has been eliminated.

Figure 9 shows a dense-graded granite mix surface and the skid data for it. The skid values shown in Figure 10 for the treaded tires are about the same as those for the sand mix and popcorn mix at higher speeds—all 3 produced skid numbers around 48 or 50 at 70 mph (112.7 km/h). For lower speeds with treaded tires, the values are

Figure 5. Bleeding in wheel path of popcorn mix.

Figure 6. Loss of aggregate in wheel path of popcorn mix.

Figure 7. Surface texture, popcorn mix.

Figure 8. Skid test data, popcorn mix.

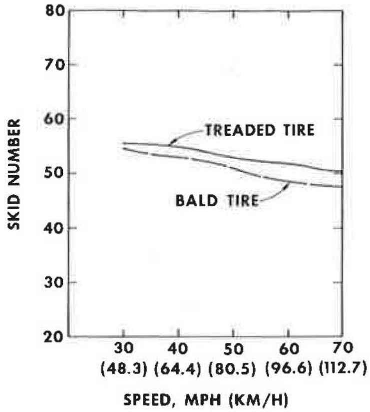


Figure 9. Surface texture, dense-graded granite mix.

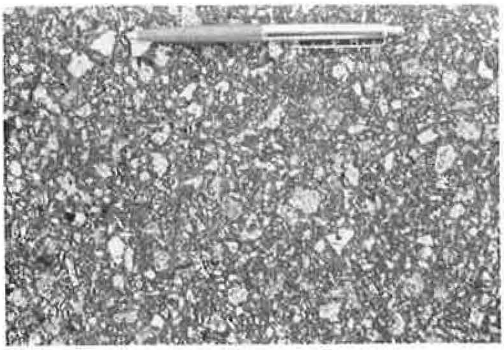
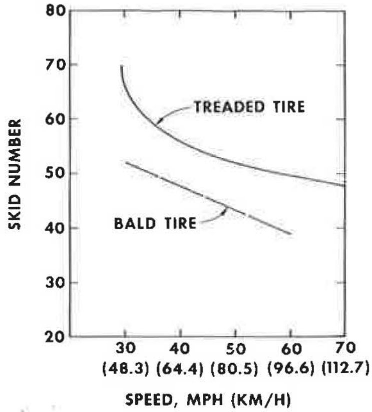


Figure 10. Skid test data, dense-graded granite mix.



somewhat lower than those for the sand mix and higher than those for the popcorn mix. The bald-tire curve demonstrates that the surface of this mix provides for the escape of water at the tire-pavement interface.

Figure 11 shows the surface of a dense-graded mix of the same gradation as the one just discussed but containing quartzite rather than granite. Figure 12 shows the skid values, which, again, are excellent. Even for the bald tire at 70 mph (112.7 km/h), the skid numbers are quite high.

The skid data for these 2 dense-graded mixes demonstrate that this type of mix can be depended on for wet traction at high as well as low speed.

Urban Mix

A second type of dense-graded mix is the urban mix. Its purpose is to withstand slow loading rates and lateral stresses found at bus stops and traffic lights. The mix design is directed toward obtaining maximum aggregate interlock and reducing the influence of the asphalt. In doing this the asphalt content is reduced to the minimum consistent with reasonable stability and density. To provide a stiffer mix, one uses a harder than usual grade of asphalt, one with a penetration grade of 60-70 or an asphalt content of 40 percent. To ensure high densities during construction, one should employ a vibratory roller. A minimum density of 92 percent MTD is specified as is the case for other dense-graded mixes.

The gradation given in Table 2 for the urban mix is very similar to that of an intermediate mix [with a top size of ¾ in. (19.05 mm)] that Virginia has used for years.

The urban mix shows a highly advantageous blending technique that employs polish-susceptible fine aggregate and polish-resistant coarse aggregate. This blending is significant economically in Virginia in that it obviates the necessity for transporting great amounts of highly skid-resistant, nonpolishing aggregates from east of the Blue Ridge, where they are plentiful, to west of the Blue Ridge, where they are scarce. The problem that occurs when mixes are made from 100 percent of some of the polish-susceptible aggregates found in Virginia is shown in Figure 13, which plots friction values versus accumulated traffic volumes for a dense-graded mix made from a polish-susceptible aggregate and for one made from a polish-resistant aggregate; the tests were conducted at 40 mph (64.4 km/h). The curve for the polish-susceptible aggregate represents a limited amount of data because Virginia outlawed the use of this aggregate in surface courses years ago. On the other hand, the curve for the polish-resistant aggregate represents data for all the bituminous surfaces in the Virginia Interstate system at the time the tests were made. The mixes made of polish-susceptible aggregates became slippery before they experienced 3 million vehicle passes; those with polish-resistant aggregates maintained a high skid value even after 25 million vehicle passes.

One of the most important things that can be inferred from this figure is what happens to the curve for the polish-resistant aggregate after it passes 25 million vehicle passes. It is believed that this curve represents enough data to conclude that its slope has leveled off and that any further decrease in friction over the expected life of the pavement surface will be insignificant. In short, this type of pavement will not become slippery before it has to be resurfaced for some other reason. So, the dense-graded asphalt concretes, which can be found on the majority of Virginia's high-class roads, are providing needed skid resistance.

The urban mix for which the surface and skid data are shown in Figures 14 and 15 is a blend of 40 percent polish-susceptible fine aggregate and 60 percent polish-resistant coarse aggregate. It has a harsh macrotexture. Unfortunately, the section for which data are shown was located at a traffic light. Therefore, tests at 70 mph (112.7 km/h) could not be performed. The tests that were made were run at night and with special precautions. Up to 60 mph (96.6 km/h) there are no differences between the bald-tire and treaded tire values, which is also true for the popcorn mix.

Another type of dense-graded mix that permits the use of polish-susceptible aggregate is the sprinkle mix. This mix, made from 100 percent polish-susceptible aggregates, is placed on the roadway and while it is still hot, but before it is rolled, is

Figure 11. Surface texture, dense-graded quartzite mix.

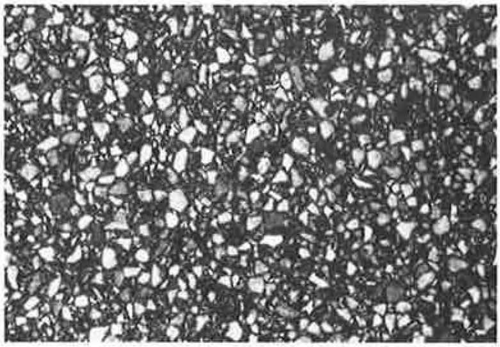


Figure 12. Skid test data, dense-graded quartzite mix.

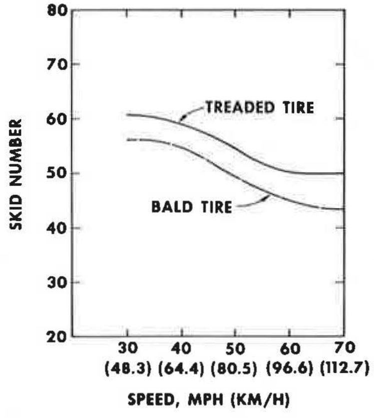


Table 2. Gradation of urban mix.

Sieve Size	Percent Passing by Weight	Sieve Size	Percent Passing by Weight
1 in.	100	No. 8	31 to 39
¾ in.	95 to 100	No. 50	6 to 14
⅜ in.	63 to 77	No. 200	2 to 6
No. 4	43 to 57		

Note: 1 in. = 25.4 mm.

Figure 13. Effect of aggregate and traffic volumes on friction values.

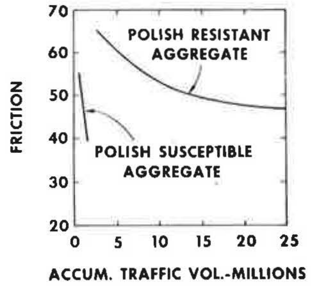


Figure 14. Surface texture, urban mix.

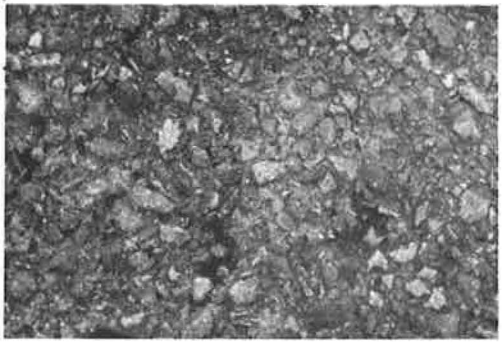
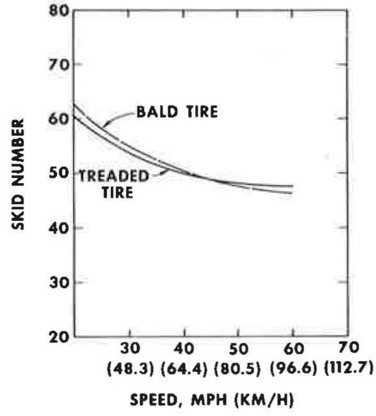


Figure 15. Skid test data, urban mix.



sprinkled with a 5-psi (34.5-kPa) application of highly polish-resistant aggregates, which are then embedded in the surface by the rollers. Unfortunately, skid data in the format used in this paper are not available for this mix, but the skid resistance has been reported to be very good (2).

CONCLUSION

In conclusion, we reiterate that Virginia has experienced a great deal of success in providing skid resistance through the use of dense-graded bituminous mixes. The 2 dense-graded S-5 mixes and the blended urban mix demonstrate that, if dense-graded mixes are designed properly and are fabricated with at least the coarse aggregate's being highly skid resistant, they can be depended on to provide excellent skid resistance throughout the range of highway speeds. Special custom mixes are needed for special situations, but the designer should employ careful design for optimum skid resistance.

REFERENCES

1. Britton, W. S. G., and Dillard, J. H. Use of Thin Sand Overlays in Virginia. Virginia Highway and Transportation Research Council, Charlottesville, Reprint 50, 1963.
2. Dillard, J. H., and Maupin, G. W. Use of a Sprinkle Treatment to Provide Skid Resistant Pavements. Virginia Highway and Transportation Research Council, Charlottesville, Reprint 81, 1971.

TECHNIQUES FOR ACHIEVING NONSKID PAVEMENT SURFACES ESPECIALLY BY DEEP TRANSVERSE GROOVING OF FRESH CEMENT CONCRETE

J.-P. Leyder and J. Reichert, Centre de Recherches Routières, Belgium

One of the research projects conducted by the Belgian Centre de Recherches Routières on slipperiness was on deep transverse grooving of fresh cement concrete. Several test sections were constructed, and their behavior was followed for over 10 years. Deep transverse grooving consists of imprinting into the pavement surface 5- to 7-mm-deep grooves, 15 to 30 mm apart. The measurements that have been carried out have brought to light the substantial increase of the sideway force coefficient (SFC) of a grooved pavement as compared to that of a smooth one. These measurements have permitted the definition of quality factors of a grooved concrete pavement. For example, the SFC drop between 20 and 80 km/h is correlated linearly to the inverse of the surface mean leveling depth, and the SFC value for a low speed depends, for practical purposes, on only the polish condition of the surface aggregate. Practical criteria have been based on this: (a) No polish-susceptible stony or sandy element may appear at the pavement surface; and (b) the average leveling depth of the grooved surface must reach at least 2 mm. Grooving may be achieved by combs or fork-like tools. A prototype machine for both grooving and spreading the curing compound has been developed and is now in use. Practical recommendations for users and machine manufacturers are offered. In addition to antiskid properties, grooved surfaces have several secondary advantages and drawbacks: (a) noticeable improvement of surface drainage, (b) reduction of dazzling glare due to the reflection of grazing light, (c) possible increased tire wear, and (d) an unpleasant whizzing noise, which could, however, be checked by a random spacing of grooves.

•ACHIEVING high and lasting pavement skid resistance, even at high speed, is becoming a primary aim of road construction. The roughness criteria to be adopted depend on the site, allowed speed, and traffic volume. Those proposed in 1957 by Giles of the British Transport and Road Research Laboratory (TRRL) are still pertinent (1, 2, 3). It is important that proved construction techniques be made available to the road engineer.

As early as 1956, the Belgian Centre de Recherches Routières initiated several research projects on roughness and the development of both cement concrete and bituminous concrete skid-resistant pavements (4, 5). The results of those investigations have been published and are reflected in official specifications. The following 5 solutions are allowed by these specifications:

1. Bitumacadams and tarmacadams (6, 7),
2. Unchipped bituminous concrete (8, 9),
3. Chipped bituminous concrete (10),
4. Chipped cement concrete (11, 12), and
5. Transversely grooved cement concrete.

Publication of this paper sponsored by Committee on Rigid Pavement Construction.

As a consequence of inadequate construction or the polishing action of traffic on polishable aggregates, the road engineer is faced with the problem of restoring the skid resistance of both bituminous pavements and cement concrete pavements. For cement concrete, mechanical procedures, such as the use of a diamond saw, may enable some roughness to be imparted to slabs that have grown slippery. Such procedures, however, are costly. They are technically efficient when the surface aggregate is hard and has a very low polish susceptibility, but they yield poor results when the aggregate has polished under traffic or when it is made of too polishable a stone.

For both cement concrete pavements and bituminous pavements, very promising techniques are being developed to restore antiskid properties by synthetic resin surfaces. Such surfaces must be waterproof, thin, rough, harsh, and lasting. Their use implies that their support and foundation are in good condition (13, 14). In addition to synthetic resins, they include a hard, little polishable aggregate. The high cost of such surfaces has been criticized. However, they appear to offer the best cost-performance ratio when use of traditional materials is difficult and when particularly exacting safety, time, or thickness requirements are imposed.

CHOICE OF A SURFACE TREATMENT TECHNIQUE FOR CEMENT CONCRETE

From 1950 to 1961, numerous manual, on-site experiments on various surface textures were carried out in Belgium. Observation of the performance of those test sections under traffic indicated that achievement of high skid resistance on wet pavement is related to 2 general features (34):

1. The presence of a rough surface texture and
2. The absence of polishable aggregate or polishable sandy elements at the concrete surface (harsh surface).

As a result of those tests, the following 2 types of surface treatment were retained:

1. Imbedding of chips at the surface of the fresh concrete (Fig. 1) and
2. Deep transverse grooving of the fresh concrete (Fig. 2).

After over 10 years of traffic wear, the test sections thus treated yielded excellent roughness values at high speed (80 km/h), as given in Table 1 (4, 15). Results noted on untreated reference sections are included in this table for comparison. The Centre de Recherches Routières started a technological research project as early as 1966 for the development and mechanization of deep transverse grooving.

Mechanical imbedding of chips in fresh concrete has recently met with renewed interest in Belgium (11, 12) because it permits use of polishable aggregates in the bulk of the concrete when chips of very low polish susceptibility are used on the surface.

PRINCIPLE OF TRANSVERSE GROOVING

Deep transverse grooving consists of creating a rough surface texture made of transverse grooves 5 to 7 mm deep and 15 to 30 mm apart (Fig. 3). In rain, these grooves provide for surface drainage and decrease the thickness of the water film between the tire and the pavement. This procedure is an efficient means of checking hydroplaning. We consider that lengthwise grooving of fresh concrete should be banned because such grooving does not provide for drainage as does transverse grooving. Furthermore, it brings on a yaw effect that is detrimental to vehicle driving and passenger safety.

PERFORMANCES ATTAINED BY TRANSVERSELY GROOVED CONCRETE

Belgium is one of the few countries in the world where transversely grooved cement concrete sections that have carried traffic for over 10 years can be found. Figure 4 shows the performance of 1 pavement. The constant gain in sideway force coefficient (SFC) displayed at 50 km/h on a transversely grooved concrete is compared to that of

Figure 1. Imbedding 10/18-size porphyry chips by tamping.

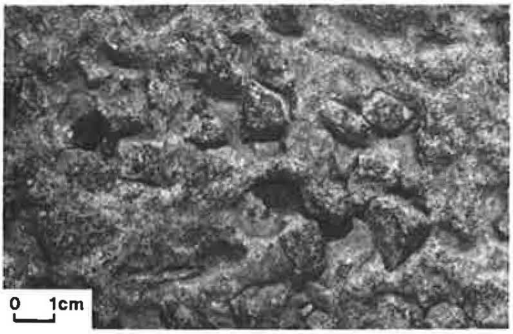


Figure 2. Tranverse grooving with a metal stand comb.

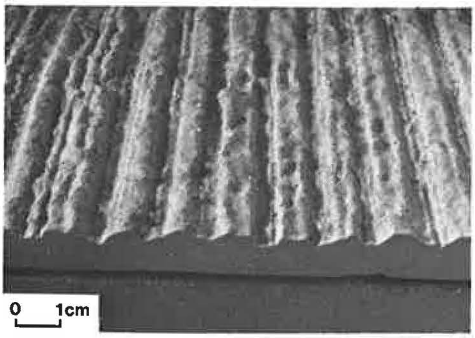


Table 1. Performance of different types of fresh concrete surface treatments.

Type	Surface Treatment	Traffic in 1970, 6 to 22 h (vpd)	Number of Years of Traffic	SFC at 80 km/h and 20 C	SFC Drop Between 20 and 80 km/h
1	Embedded chip	2,800	13 to 16	0.53 to 0.64	0.24 to 0.30
2	Smooth reference section	2,800	16	0.40	0.41 to 0.45
3	Transverse grooving	10,700	11	0.55 to 0.75	0.04 to 0.27
4	Smooth reference section	10,700	11	0.42 to 0.50	0.29 to 0.42
5	Reference section with polish-susceptible aggregate	12,100	13	0.25 to 0.44	0.22 to 0.34

Figure 3. Deep transverse grooving with 6-mm-diameter metal forks, 20 mm apart.

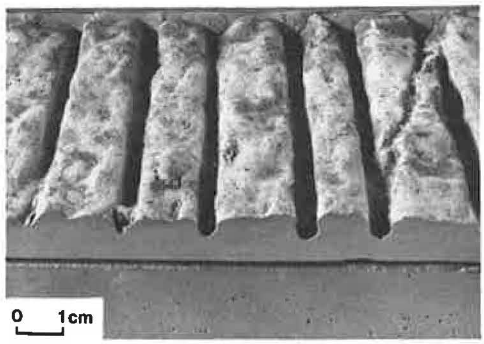
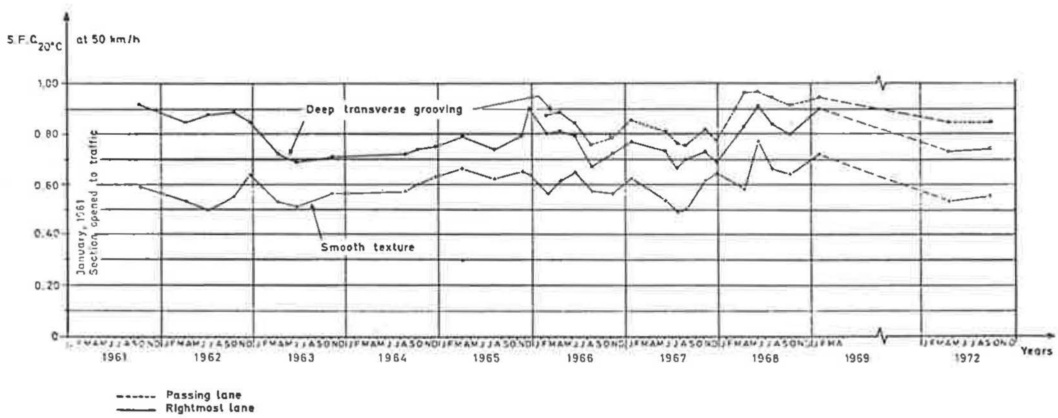


Figure 4. Evolution of sideway force coefficient at 50 km/h of 2 identical pavements, one deeply grooved, the other smooth.



a concrete with a smooth surface. SFC on wet pavement was measured with a TRRL odoliograph outfitted with a smooth tire. A correction was applied for conventional reduction of the measured value to a water temperature on the pavement of 20 C (3). Investigations also brought out the influence exercised on SFC by the difference in weight of the traffic (15, 16). The SFC values measured in the far right lane and passing lane of an expressway are shown in Figure 4 (86 percent heavy vehicles were in the far right lane and 14 percent were in the passing lane).

QUALITY FACTORS OF TRANSVERSE GROOVING

Systematic analyses of SFC values at different speeds (20, 50, and 80 km/h) have brought to light 3 facts (17).

1. The larger the mean leveling depth, H, of the grooved texture [sand patch test, (18)], the smaller the SFC drop between 20 and 80 km/h at any time during the life of the pavement. The linear regression equation between these 2 variables reads, in the interval 0.40 mm < H < 3.0 mm,

$$\Delta SFC_{20-80\ km/h} = \frac{23}{H} + 2$$

H is expressed in millimetres and ΔSFC is expressed in hundredths of unity (19).

The correlation coefficient obtained by putting together 43 measured pairs of values from 3 different grooved sections was found to exceed 99.9. This relationship is independent of the state of polishing of the pavement.

2. The SFC value at 20 C at a speed of 25 to 30 km/h is for practical purposes independent of H; it depends mainly on the polishing condition of the elements present at the pavement surface at the time of measurement. This is shown by Figure 5 which compares the SFC values of a grooved concrete and a smooth concrete at different speeds after 3 years and 10 years of traffic wear. The poor results obtained at 80 km/h as a consequence of a smooth surfacing should be stressed.

3. For pavements with identical large-scale surface texture, the rate of decrease of the SFC, at any speed, under the action of traffic, is larger or smaller according to whether the pavement surface contains or does not contain polish-susceptible elements (Fig. 6).

CRITERIA FOR A LASTING TRANSVERSE GROOVING

The practical application of the aforementioned rules has led to the formulation of execution criteria aimed at ensuring the following performances after many years of traffic wear: (a) SFC value at 20 C and 80 km/h larger than 0.50 and (b) SFC drop between 20 and 80 km/h of about 0.15 to 0.25. The criteria are fine- and large-scale texture. For fine-scale texture, no polish-susceptible stony or sandy element may appear at the pavement surface. Therefore, the surface aggregate must have a high enough polished stone value (PSV) (20). The required value depends on the site, allowed speed, and traffic volume (21, 22). Sands made from a polish-susceptible material must be banned. For large-scale texture, the grooves must be 15 to 30 mm apart, and their depth must be such that, after early trimming of the grooves under traffic, H is at least 2 mm. This should meet the requirement for a residual H of at least 1 mm after 2 or 3 years of traffic wear. These performances have been achieved with the conventional Belgian concrete mix design, which is relatively dry, rich in cement, and rich in mortar compared to concrete mixes used in other countries. That is to say

$$W/C \approx 0.42$$

$$C = 375 \text{ to } 400\ kg/m^3$$

$$C + S = 805 \text{ to } 825\ kg/m^3$$

Figure 5. Sideway force coefficient values.

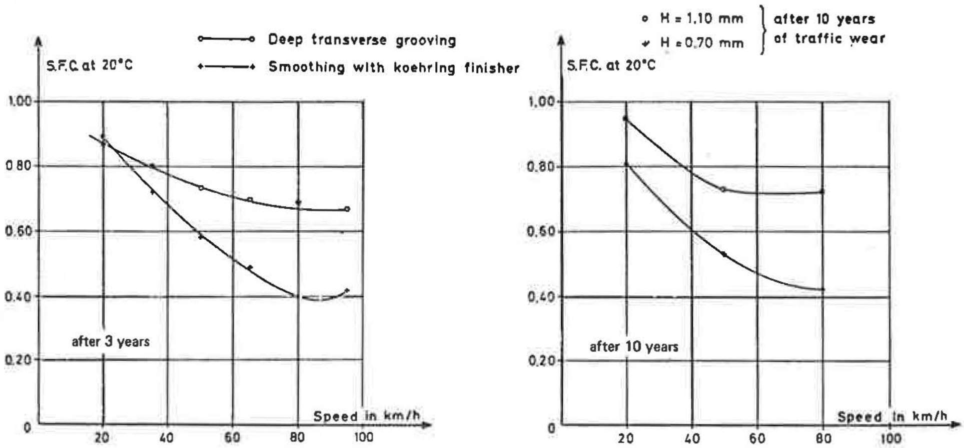
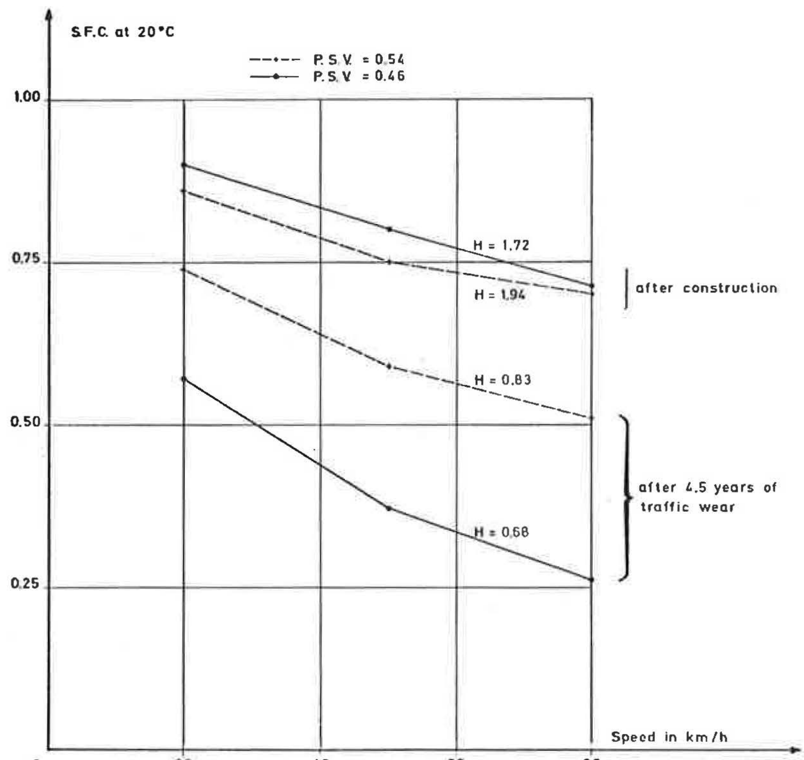


Figure 6. Influence of polishing of surface aggregate on evolution of slipperiness with time.



where

C = kilograms of cement per cubic metre of fresh concrete on the site,
S = kilograms of sand per cubic metre of fresh concrete on the site, and
W = litres of water per cubic metre of fresh concrete on the site.

The practical execution of the grooved concrete must comply with current expertise for choice of materials, mix preparation, application on the site, and grooving and curing of fresh concrete.

CHOICE OF A GROOVING TOOL

Choosing an adequate grooving tool is essential to achieve an effective transverse grooving. The tool must groove the fresh concrete without appreciably disturbing the surface or pulling stones away. Two types of grooving tools were selected, namely,

1. Fork-like grooving tools (Fig. 7) made of pivoted curved metal teeth, 4 to 6 mm in diameter, 15 to 30 mm apart, individually loaded with 300 to 500 g each, that imprint transverse grooves 5 to 7 mm deep into the mortar (Fig. 3); and
2. Combs made of a single row of bundles of metal or PVC strands, 15 mm apart (Fig. 8). Each bundle, ballasted with 30 to 50 g, consists of 10 strands 65 to 75 mm long. The grooves obtained with such combs are somewhat less regular and shallower than those made by the fork-like tools, but the texture of the mortar is harsher than in the case of fork-made grooves (Fig. 2).

Whatever the type of grooving tool used, some authorities feel that it is advisable to randomly space the grooves (23).

MECHANIZATION OF TRANSVERSE GROOVING

Various machines for grooving and spreading the curing compound have been at work for several years in Belgium. The first ones were the outcome of the development of a prototype conceived in 1967 by the Centre de Recherches Routières in collaboration with a group of contractors (24) (Fig. 9). The wide experience gained on numerous works from 1967 to the present enabled us to specify several criteria to benefit practitioners and machine manufacturers (19). We recommend study of the possibility of conveniently exchanging various rational tools, adjusting grooving pressure, carrying out a repeated grooving at the same place, devices for avoiding nozzle clogging and sedimentation of the curing compound pigments, existence of safety devices, possibility of controlling the spray flow, automatization of the full cycle (grooving and curing compound spraying), ease of displacement on the works, quick extension of the machines to different concreting widths, and progression speed of the grooving process being appreciably larger than the average concreting speed.

PRACTICAL POPULARIZATION OF TRANSVERSE GROOVING IN BELGIUM AND EUROPE

The early mechanical executions of transverse grooving data back to 1967. From 1968 to 1970, grooved pavements increased in number in Belgium; since January 1, 1970, a standard specification for state highways (25) has prescribed the method of generalized application of the mechanical transverse grooving of fresh concrete. This requirement has resulted in the execution of 4 800 000 m^2 of grooved expressway pavement, 1 350 000 m^2 of state highways, and 45 000 m^2 of airfield runways. The present cost of fresh concrete transverse grooving varies between 1.70 and 7.00 Belgian francs per m^2 (0.04 to 0.16 U.S. dollars).

Apart from Belgium, which, at present, is the most important concrete road builder in Europe, transverse grooving has also spread out in France, in Great Britain, and in Denmark. Spain also is turning to this technique for its new cement concrete highways program. In Great Britain, several experiments of manual transverse grooving have been undertaken since 1964 (16). Since 1969, contractual rules of execution and acceptance of fresh concrete transverse grooving are enforced (26). In France, the first

Figure 7. Fork-like grooving tool made of metal teeth 6 mm in diameter, 20 mm apart.

Figure 8. Metal strand comb with bundles 15 mm apart.

Figure 9. Grooving and curing-compound-spraying machine.

Figure 10. Reflection of vehicle headlights at night on a lightly brushed surface (background) and a deeply grooved surface (foreground).

deep transverse grooving executions on the Montpellier-Nîmes expressway date back to 1968 (27). They were followed by others on both airfield runways and expressways; in 1969, the Ministry of Equipment and Housing issued specifications about the texture depths for grooved pavements (28). In Denmark, executions done after 1969 were inspired by the British experience in grooving (29).

SECONDARY ADVANTAGES AND DRAWBACKS OF DEEP GROOVING

Deep grooving displays some secondary advantages and drawbacks of which the road engineer must be fully aware.

Behavior In Rain

As compared with a smooth texture, a definite reduction of the fine water spray behind trucks is noticed. On the other hand, after rain the pavement remains wet for a longer time between the grooves although it retains nonskid characteristics that are well above those of a wet, smooth concrete (27).

Dazzlement

Troublesome mirroring, which is particularly felt on a wet pavement, at night, or at sunset and sunrise, is strongly reduced by the presence of deep transverse grooves (Fig. 10). Road lighting experts consider, though, that the dulling aspect of grooved pavements requires an increase of the electric power required to obtain the same lighting.

Tire Wear

It has been contended that tire wear would be increased on grooved pavements, but, to our knowledge, no systematic measurements have been carried out in this respect. If the contention is true, the expenses incurred for a slightly more frequent replacement of tires should be compared to the resulting improvement of driver safety.

Studded Tires

In Belgium, the use of studded tires dates back to 1967-1968. Although no systematic observations have been made, no detrimental action of studded tires on cement-rich grooved concretes has been reported.

Noise

The problem of noise has been much debated. It must be acknowledged that, especially on newly built pavement and some regularly patterned or sine-wave-shaped groovings, an inconvenient whizz can be heard, particularly with radial ply tires. To avoid this, randomly spaced grooving, as advocated by Weaver (23), seems to be commendable. On the total set of numerous grooved sections constructed since 1961, the average noise has been found to be similar to that produced by other deep, rough texture pavements such as chipped cement, asphalt concrete, or precoated chips (30). The noise level produced on dry, grooved pavements exceeds that on smooth and polished pavements. In rainy weather, the situation is reversed because, on smooth pavements, finely sprayed water is stirred up by passing vehicles.

Riding Quality

This is an important area in which the experts are only now trying to define basic criteria (31). The only method at our disposal is the comparison of two 1 100-m sections differing only by type of surface treatment—either with deep transverse grooving or with no grooving at all. The comparison of the evenness of these 2 sections by means of the viagraph discloses no significant difference (32). On some recent expressway sections, however, obvious signs of inadequate comfort and safety have been reported by users. Their natural reaction was to look at the pavement and infer that the grooved surface was the cause. Actually, scrutiny by the Belgian roads authority disclosed a systematically wave-like surface duplicating the pattern of the reinforcing mesh (33).

GENERAL CONCLUSIONS

After over 12 years of experience with cement concrete roads treated by deep transverse grooving of the fresh concrete, we developed a technically and economically satisfactory solution that makes it possible to retain, after many years of heavy traffic, a high and lasting skid resistance on wet pavements, even at high speed. A trend is now developing in several countries toward burdening road engineers with civil and penal responsibility for some traffic accidents. They must be aware of the necessity of improving compromises and of the field still open to research for better future control of nuisance factors such as noise and lack of riding quality.

ACKNOWLEDGMENTS

We would like to thank our colleagues at the Public Works Ministry and the Belgian contractors for their active collaboration in the construction of test sections; the task groups of the Centre de Recherches Routières for the encouragement and advice received during the research; our coworkers, G. Van Heystraeten, J. De Coster, P. Vandevelde, J. Hermans, and N. Damm and his team, for their help during the experiments; L. Welter, A. de Henau, J. Huet, J. E. Romain, and J. Van de Vijver for their constructive suggestions during the drafting of this paper; and the Institut pour l'Encouragement de la Recherche Scientifique dans l'Industrie et l'Agriculture for its financial support of this project.

REFERENCES

1. Giles, C. G. The Skidding Resistance of Roads and the Requirements of Modern Traffic. Proc. Inst. Civil Engineers, London, Vol. 6, 1957, pp. 216-249.
2. Critères de Qualité des Matériaux Pierreux. 9th Congress of Association Permanente des Congrès Belge de la Route, Liège, Belgium, Rapport A.1/4 de la Commission des Critères, 1959.
3. Daube, J. Coefficient de Frottement Transversal et Pourcentage d'Accidents dus à la Glissance. Technique Routière 2/4, 1957, pp. 182-184.
4. Daube, J. La Glissance des Revêtements de Chaussées (II)—Facteurs Influençant le Coefficient de Frottement Transversal. Centre de Recherches Routières, Brussels, Publication F.30/58, 1958, pp. 24-25.
5. Daube, J. L'Etude de la Glissance des Revêtements de Chaussées au Centre de Recherches Routières. Technique Routière 4/1, 1959, pp. 4-25.
6. Outer, P., Reichert, J., and Hubrecht, L. Compositions de Tarmacadams et Bitumacadams. Technique Routière 13/2, 1968, pp. 1-8.
7. Reichert, J. Quelques Progres Récents Dans le Domaine des Revêtements Hydrocarbonés. Journée du Bitume, Association Permanente des Congrès Belge de la Route, Antwerp, Belgium, June 11, 1968.
8. Outer, P. Bitumes et Bétons Asphaltiques. Technique Routière 12/4, 1967, pp. 1-19.
9. Huet, J. Enseignments Tirés des Routes Expérimentales de Bétons Asphaltiques (Grand-Manil, Meise, Hoeilaart). Technique Routiere 17/1, 1972, pp. 3-26.
10. Van Heystraeten, G. Sections Expérimentales en Béton Hydrocarboné Clouté à Hoeilaart. Technique Routiere 16/2, 1971, pp. 27-56.
11. Van Heystraeten, G. Connaissances Acquises en Matière de Béton de Ciment Clouté. Technique Routière, 1974.
12. Reichert, J. Routes en Béton de Ciment de Demain: Conception, Construction et Entretien. 2nd Symposium Européen des Routes en Béton, Berne, Switzerland, Vol. 2, 1974.
13. Hubrecht, L. Revêtements Routiers Minces à Base de Résines Synthétiques—Etude d'Information et Premiers Résultats Expérimentaux. Centre de Recherches Routieres, Compte Rendu de Recherche CR/3/72, Brussels, 1972.

14. Hubrecht, L. Sections Expérimentales de Revêtements Résineux Anti-Dérapants sur Béton de Ciment et sur Béton Bitumineux—1970-1971. Technique Routière 17/4, 1972, pp. 17-44.
15. Leyder, J.-P. Synthèse sur la Rugosité des Bétons de Ciment. Technique Routière 13/3, 1968, pp. 1-16.
16. Leigh, J. V., Burks, A. E., Davis, R. F. P., Gate, D. D., Nicholas, J. H., Sharp, D. R., and Williams, O. T. Chaussées en Béton de Ciment. 14th World Congress, Association Internationale Permanente des Congrès de la Route, Prague, Question 3-8, 1971, pp. 19-21.
17. Leyder, J.-P. Skidding Resistance of Cement Concrete and Mechanization of the Transversal Grooving, Tome 2. Internat. Colloquium on the Interrelation of Skidding Resistance and Traffic Safety on Wet Roads, Berlin, 1968, pp. 623-632.
18. Mode Opératoire—Essai à la Tache de Sable—Détermination de la Profondeur Moyenne de Nivellement des Stries des Revêtements en Béton de Ciment. Centre de Recherches Routières, Brussels, Méthode de Mesure MF/32/69, 1969.
19. Leyder, J.-P. Dix Ans d'Expérience de Striage Transversal Profond du Béton a l'Etat Frais. 2nd Symposium Européen des Routes en Béton, Berne, Switzerland, 1973, pp. 311-323.
20. Determination of the Laboratory Determined Polished-Stone Value, Methods for Sampling and Testing of Mineral Aggregates, Sands, and Fillers. British Standards Institution, BS 812, 1967, pp. 88-97.
21. Mechanical Properties, Specification for Gravel Aggregates for Surface Treatment (Including Surface Dressings) on Roads. British Standards Institution, BS 1948, 1967, p. 7.
22. Circulaire sur les Chaussées Glissantes. Bulletin de Liaison du Laboratoire des Ponts et Chaussées, Paris, No. 39, 1969, p. 56.
23. Weaver, J. Striage Profond de Routes en Béton. 2nd Symposium Européen des Routes en Béton, Berne, Switzerland, 1973, pp. 293-309.
24. Leyder, J.-P. Machine Pour le Striage Transversal et la Pulvérisation de "Curing Compound" sur le Béton Frais. Technique Routière 13/1, 1968, pp. 3-23.
25. Administration des Routes. Cahier Général des Charges-Type 108 (1967 ed.), Ministère des Travaux Publics, Brussels, 1970.
26. Specification for Road and Bridge Works. Her Majesty's Stationery Office, London, 1969, pp. 93-94.
27. Arribehaute, M., Reverdy, M., Gaud, M., Parey, M., Slama, M., and Aileret, M. Chaussées en Béton de Ciment (Plisson, Coordinateur). 14th World Congress, Association Internationale Permanente des Congres de la Route, Prague, Question 3-7, 1971, pp. 19-22.
28. Rouyer, F. Essais de Striage Transversal des Chaussées en Béton de Ciment. Bulletin de Liaison dus Laboratoire des Ponts et Chaussées, Paris, No. 56, Dec. 1971, pp. 79-81.
29. Johansen, T., and Sloth, S. Chaussées en Béton de Ciment. 14th World Congress, Association Internationale Permanente des Congrès de la Route, Prague, Question 3-5, 1971, pp. 7-8.
30. Maynard, D. P., and Lane, F. E. Road Noise With Particular Reference to Grooved Concrete Pavements. Cement and Concrete Assn., London, ITN 2, Aug. 1971.
31. Leger, M. L'Uni des Revêtements. Comité de Glissance, 14th World Congress, Association Internationale Permanente des Congrès de la Route, Prague, Chapter 9, 1971, pp. 139-153.
32. Influence du Striage des Revêtements en Béton sur les Mesures au Viagraphe. Technique Routière 16/2, Question 12, 1971, pp. 57-62.
33. Dechamps, Y., Doyen, A., Dutron, P., Hofmans, M., Van Ael, P., and Van Loocke, A. Réalisation à Grande Echelle de Revêtements Routiers en Béton Armé Continu. 2nd Symposium Européen des Routes en Béton, Berne, Switzerland, 1973, pp. 104-109.
34. Sabey, B. E. Principles de l'Interaction Entre les Propérietés du Pneumatique et la Texture Superficielle de la Chaussée. Comité de Glissance, 14th World Congress, Association Internationale Permanente de Congrès de la Route, Prague, Chapter 2, 1971, p. 13.

SPONSORSHIP OF THIS RECORD

LEGAL RESOURCES GROUP COUNCIL
Joseph M. Montano, Colorado Department of Highways, chairman
Joseph D. Buscher, Saul C. Corwin, Robert W. Cunliffe, Clifton W. Enfield, David R. Levin, Harrison Lewis, Daniel R. McLeod, Louis R. Morony, Ross D. Netherton, Asa C. Richardson, David E. Wells

GROUP 2 COUNCIL—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
W. B. Drake, Kentucky Department of Transportation, chairman
John L. Beaton, John A. Deacon, Arthur L. Elliott, L. F. Erickson, W. J. Halstead, Moreland Herrin, Wm. P. Hofmann, Donald R. Lamb, Thomas D. Larson, R. V. LeClerc, D. W. Loutzenheiser, Eugene B. McDonald, Carl L. Monismith, Howard H. Newlon Jr., Gordon K. Ray, Robert D. Schmidt, M. D. Shelby, F. W. Thorstenson, Eldon J. Yoder

PAVEMENT DESIGN SECTION
Carl L. Monismith, University of California, Berkeley, chairman

Committee on Surface Properties-Vehicle Interaction
W. E. Meyer, Pennsylvania State University, chairman
Glenn G. Balmer, Frederick E. Behn, Andrius A. Butkunas, A. Y. Casanova, III, Michael W. Fitzpatrick, William Gartner, Jr., Ralph C. G. Haas, Douglas I. Hanson, Don L. Ivey, William F. Lins, David C. Mahone, Paul Milliman, Alexander B. Moore, E. W. Myers, Arthur H. Neill, Jr., Bayard E. Quinn, John J. Quinn, Frederick A. Renninger, Rolands L. Rizenbergs, Hollis B. Rushing, Richard K. Shaffer, George B. Sherman, Elson B. Spangler, James C. Wambold, M. Lee Webster, E. A. Whitehurst, Ross G. Wilcox, Dillard D. Woodson

BITUMINOUS SECTION
Moreland Herrin, University of Illinois, chairman

Committee on Characteristics of Aggregates and Fillers for Bituminous Construction
James M. Rice, Federal Highway Administration, chairman
J. T. Corkill, Sabir H. Dahir, Charles R. Foster, Bob M. Gallaway, Richard D. Gaynor, John M. Gibbons, William H. Gotolski, Donald W. Lewis, Robert P. Lottman, Charles R. Marek, G. W. Maupin, Jr., W. E. Meyer, Frank P. Nichols, Jr., Robert E. Olsen, Vytautas P. Puzinauskas, Garland W. Steele, Egons Tons

CONSTRUCTION SECTION
Robert D. Schmidt, Illinois Department of Transportation, chairman

<u>Committee on Rigid Pavement Construction</u>
Sanford P. Lahue, U.S. Department of Transportation, chairman
Woodrow J. Anderson, C. E. Aten, Kenneth J. Boedecker, Jr., Don Cahoone, John E. Eisenhour, Jr., Anthony J. Fusco, Gene K. Hallock, Harold J. Halm, Emmett C. Kaericher, Jr., John V. Kelly, Robert J. Lowe, Sr., T. E. McElherne, Alvin H. Meyer, Issam Minkarah, August F. Muller, W. Lee Powell, William G. Prince, Jr., Gordon K. Ray, Robert D. Schmidt, Earl R. Scyoc, Peter Smith, Donald L. Wickham, James E. Wilson, Jr., William A. Yrjanson

William G. Gunderman, Lawrence F. Spaine, and John C. Vance, Transportation Research Board staff

Sponsorship is indicated by a footnote on the first page of each report. The organizational units and the chairmen and members are as of December 31, 1973.