SPECIAL HRB REPORT

95

State of the Art:

Rigid Pavement Design
Research on Skid Resistance
Pavement Condition Evaluation

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HIGHWAY RESEARCH BOARD

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1968

2101 Constitution Avenue   Washington, D. C. 20418

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Subject Area

25 Pavement Design
26 Pavement Performance

N.R.C. HIGHWAY RESEARCH BOARD

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Foreword

As in all other areas of development of civilization, the highway engineer finds it desirable to pause periodically and reflect on the circumstances which have permitted him to advance to the point on which he now stands. His future course is largely determined by the foundation already constructed and the measure of success which has been his in certain avenues of approach to his particular problem.

During the Annual Meeting of the Highway Research Board in 1965, need for a review of the art of pavement design and a restatement of research required to further the advancement of pavement design technology was recognized by the Pavement Division of the Department of Design. All committees were requested to undertake the formidable task of evaluating the present state of the art and future research needs in the general area of interest of each respective committee. This report contains the summaries of the efforts of three of these committees.

The first paper was prepared by a subcommittee of the Committee on Rigid Pavement Design. It compiles a general summary of the most significant work done in the past half century. No particular attempt has been made to judge this work, but substantiating evidence and subsequent research are reported whenever possible. An extensive bibliography has been provided for those who wish to explore the subject more deeply.

The second paper, sponsored by the Committee on Surface Properties—Vehicle Interaction, was written by T. I. Csathy, with the cooperation of committee members M. D. Armstrong and W. C. Burnett. The paper summarizes the state of the art in the field of skid resistance research. The report presents major conclusions from investigations conducted in the past four decades, outlines present trends in skid resistance research, and summarizes some of the major research needs.

The final paper was prepared by a subcommittee of the Committee on Pavement Condition Evaluation. The paper discusses the problem of pavement condition evaluation along with some of the major work that has been accomplished in the past, offers some ideas about the present situation, and suggests areas for future research.

Acknowledgment of the efforts of individuals making major contributions to this report is shown in the separate papers.

Contents

State of the Art of Rigid Pavement Design

THE FIRST extensive concrete road system in the United States was constructed in Wayne County, Michigan, in 1909. At that time there was no rational design theory in existence and consequently, design decisions were based entirely on "engineering judgment." Since that time civil engineers have continued to grapple with the many problems involved in the design of such pavement. Goldbeck (1) and Older (2) independently developed formulas for approximating the stresses in concrete pavements in the early 1920's. The best known of these formulas is generally called the "corner formula" and it was the basis for rigid pavement thickness design for many years. In 1926 Westergaard completed his treatise on the analysis of stresses in pavement slabs (3). The Westergaard equations have become the definitive design equations for pavement slabs in the United States.

Although many approaches have been made to the solution of this problem, limitations of conventional mathematics, particularly hand solutions, have restricted developments. Owing to the complexity of the problem all of the solutions involve severely limiting assumptions which bypass certain influencing factors that, in reality, are very important. As a consequence, none is completely satisfactory.

Several large-scale road tests have been conducted in attempts to bridge this gap between theory and reality. These include the Bates Road Test in 1922, the Maryland Road Test in 1950, and the AASHO Road Test in 1958-61. All three of these full-scale experiments have added to the knowledge of pavement design. The AASHO Road Test was large enough to provide significant information, but even it considered only six basic variables: (a) slab thickness, (b) slab length, (c) axle load, (d) number of axle repetitions, (e) subbase thickness, and (f) jointed reinforced vs jointed plain pavements.

In addition to these road tests, several large tests have been carried out by the U.S. Corps of Engineers for the U.S. Air Force, such as the Sharonville Test Tracks. The results of these tests have been particularly pertinent to airfield pavements but also shed additional light on highway pavement problems.

Many engineers recognized the magnitude of the problem. The Bureau of Public Roads as well as many state highway departments recognized the importance of the influence of environment and climatic factors on the performance of pavements. As a result there was a proliferation of small research projects, many of them involving "in-service" pavements. Each of these projects was naturally limited in size and therefore limited in the variables it considered. Furthermore, a tremendous variation in materials and testing methods has made it difficult to correlate the resulting information.

Pavement design involves four general classes of variables: (a) load variables, (b) structural variables, (c) regional variables, and (d) performance variables. The "Guidelines for Satellite Studies of Pavement Performance" (4) discusses these variables in some detail. Major theoretical efforts have been directed toward evaluating structural variables, and the fact that they interact prohibits the consideration of all variables in any single test.

To further complicate the problem, many new techniques have been developed in recent years. Continuously reinforced concrete pavements have been investigated and are currently being used. Continued improvements are being sought in the use of prestressed concrete pavements. Other recent developments involve the use of new materials such as expansive cements and synthetic aggregates. In addition, traffic volumes and axle or airplane gear weights have increased. The advent of these new materials and this increased traffic makes the continued improvement and up-dating of pavement design techniques desirable.

All engineering developments combined theory and experience in varying degrees. These developments then lead to design procedures for "current use" but always leave many questions unanswered. A great deal of research is presently in progress toward solving these problems. A critical examination of this research in light of the overall problem and program would be a valuable addition to current techniques.

After considering the overall problem, the work that has been done and that is currently being done, it is valuable to consider the additional research needed. This undoubtedly would include some basic research, some applied research and, in certain instances, some "far-out concepts" which may be very valuable.

In this report four basic types of portland cement concrete pavement will be considered. These are (a) plain concrete pavement, (b) jointed reinforced concrete pavement (c) continuously reinforced concrete pavement, and (d) prestressed concrete pavement. These categories will be carried through each succeeding chapter where appropriate.

In the preparation of this report every effort was made to avoid expressing personal opinions of the authors. The statements made are intended to reflect the information contained in the various references or a reasonable interpretation of them. There was no intention to imply that one design or practice is considered superior to another unless the reference material was cited to support the statement. The reader is urged to personally verify any such statements contained here by consulting the reference item.

HISTORY OF CONCRETE PAVEMENT

Prior to 1890, concrete as known today was not used for building roads (5). In the early 1890's three important "firsts" occurred. In 1891 the first portland cement concrete pavement was constructed at Bellefontaine, Ohio. It consisted of an 8-ft-wide strip of concrete to provide a solid pavement in front of a line of hitching rails (6).

In 1893 Bellefontaine, Ohio, constructed the first full-width concrete pavement, and J. F. Duryea successfully operated the first American-made gasoline-engined automobile (7). Within the following half century paved roadways and motor vehicles became vitally important in the United States.

From 1893 until 1908 less than five miles of concrete pavement were constructed in the entire country, and these were located on city streets or private estates. In 1909 the first mile of rural concrete pavement in the United States was constructed in Wayne County, Michigan, and opened to public traffic on July 4th (8). Approximately four miles of rural road were paved with concrete that year. The mileage of rural concrete roads increased annually; the first big increase came in 1912 when 250 miles were constructed. By 1924 over 31,000 miles of concrete pavement were in use, and construction was proceeding at the rate of 6,000 miles per year (9). During this same period, 1909 to 1924, the production of motor cars increased from about 350,000 to over 3,300,000 per year, and the number registered went from about 306,000 to over 15,400,000 (10).

Neither the vehicle nor concrete pavement is the product of a single inventor; rather they have been developed through the efforts of thousands of technical and nontechnical people. Today's concrete pavements are the result of the accumulated experience of pavement engineers gained by (a) study and appraisal of existing pavements, (b) observation of trial roads and long-range experimental road tests carrying normal traffic, (c) accelerated controlled traffic tests on existing or specially constructed pavement sections, (d) laboratory experimentation, and (e) theoretical and rational analyses.

Between 1893 and 1920 there was very little basic technical information available to highway engineers concerned with concrete pavements. The experimental roads of the period were constructed primarily to determine the most economical designs applicable to local climate, traffic and subgrade conditions (11).

The large-scale road building program which followed World War I intensified the need for engineering data. For example, no test information was available on the supporting strength of subgrade soils, stresses and deflections induced by axle loads of moving vehicles, or effects of temperature and moisture variations on the performance of the pavement (12). Consequently, a nationwide program of research was launched in 1920 by the Bureau of Public Roads and the Highway Research Board. Many universities and state highway departments participated in this endeavor.

Between 1920 and 1923 the State of Illinois constructed and operated the Bates Experimental Road at Bates, Illinois. This was the first controlled traffic road test, and from it came the first design criteria for determining the thickness of a concrete pavement for a known wheel load (2). Other outstanding road tests were conducted in the 1920's and 1930's. The Pittsburg, California, Road Test, started in 1921 and completed in 1922, was designed to determine the efficiency of both reinforced and nonreinforced pavements of varying designs. Although the results were not conclusive, they added to the supply of technical information. They also indicated that longitudinal joints were effective in preventing longitudinal cracks (12).

From 1930 through 1936 the Bureau of Public Roads conducted the Arlington Test. This was an extensive investigation of the structural design of concrete pavement made at Arlington, Virginia. This research work supplied the basis for modern concrete pavement design criteria (13).

In the middle 1930's the use of deicing salts for winter maintenance produced severe scaling of concrete pavement surfaces in the northern states. Numerous road tests were constructed containing various ingredients that might produce scale-resistant pavements. Air-entrained concrete came out of this work. The earliest use of air-entrainment was in New York State in 1938. Between 1938 and 1942 a total of 17 test pavements were constructed in several northern states.

In the late 1930's many highway engineers became concerned about the use of expansion joints where contraction joints were also used. The use of dowels in closely spaced contraction joints and other problems in the joining of pavements were also studied in view of service records. To provide answers to these questions the Bureau of Public Roads authorized the construction of long-range experimental road tests in California, Kentucky, Michigan, Minnesota, Missouri and Oregon (14).

Just prior to World War II, pumping became common on primary systems in many sections of the country. In most cases the pumping developed on roads that carried large volumes of heavily loaded vehicles, and in areas where subgrade materials were mostly claylike or plastic in character. Many pavements on major truck routes were damaged by pumping, while pavements on lightly traveled roads with comparable designs, ages, and subgrade soils did not pump (15). Pumping is usually not as serious a problem in connection with airport pavements.

During World War II there was a significant increase in the volume and weight of truck traffic on highways and a large increase in the wheel load and tire pressure of military aircraft on airport pavements. In addition, adequate maintenance during this period was almost impossible, leaving many of these pavements in serious need of repair and upgrading. Added to this was the need for many miles of additional pavements to satisfy the tremendous growth of both rural and metropolitan areas. Consequently, research programs in all phases of pavement technology were intensified to meet the ever-increasing demands of postwar traffic. Two of these programs are outstanding—Maryland Road Test One-MD and the AASHO Road Test.

The Maryland Road Test One-MD was conducted on a section of existing concrete pavement near La Plata, Maryland, in the summer of 1950. The principal objective was to determine the effects of loads up to 44,800 lb per tandem axle on this pavement. This was the first controlled traffic test made on concrete pavement since the Bates and Pittsburg road tests in 1921-1923 (16).

The AASHO Road Test, sponsored by the American Association of State Highway Officials and other highway agencies, was conducted on a specially constructed section of highway near Ottawa, Illinois, between 1958 and 1961. One of the objectives of the AASHO Road Test as stated by the National Advisory Committee was "to determine the significant relationships between the number of repetitive applications of specified axle loads of different magnitude and arrangement and the performance of different thicknesses of uniformly designed and constructed asphaltic concrete, plain portland cement concrete, and reinforced portland cement concrete surfaces on different thicknesses of bases and subbases when on a basement soil of known characteristics." As far as portland cement concrete pavements are concerned, six basic variables were considered: (a) slab thickness, (b) slab length, (c) axle load, (d) number of axle repetitions, (e) subbase thickness, and (f) presence or absence of steel reinforcement.

Pavement Foundation

The need for uniformity of subgrade support under concrete pavements has long been recognized. Specifications published in 1910 contain a section relating to the preparation of the subgrade (17). However, it was not until 1924-1925 that rapid field methods of identifying the quality of subgrade soils became available (18). Prior to 1925, acceptability of subgrades depended entirely on the judgment of highway engineers. The publication of the Rose reports in 1925 marked a turning point in highway and concrete pavement design. They became the foundation upon which much subsequent subgrade soil research was based. These reports also pointed out that studies were needed to evaluate the influence of subgrade soils on pavement performance. However, it was not until the early 1930's that serious attention was given to such studies (19).

Subbases were used under concrete pavements as early as 1894, when alleys in Boston were constructed following sidewalk procedures (8-in. cinder subbase under 5-in. concrete). One of the first uses of subbases under street pavements was in Richmond, Indiana, in 1896. It consisted of 10 in. of compacted stone rubble under 5-in. and 6-in. thick concrete pavements (20). Specifications published in 1910 contain sections pertaining to subbases. It was specified that subbases "... of clean, hard, suitable material, not exceeding 4 in. in largest dimension ... be used ... where required; that they have a minimum thickness of 6 in. and "be thoroughly rolled and tamped" (17). The use of subbases to overcome the effect of poor subgrade soils was recommended by Rose in 1925 (18). By the end of World War II the majority of states were specifying subbases to prevent pumping and to aid in maintaining the structural capacity of pavements (21). However, this was not a common practice for city street and airfield design.

Pavement Thickness

A tabulation of data on 29 concrete roads constructed in Ohio in 1911, 1912, and 1913 shows that a wide variety of thicknesses were used. Those with uniform cross sections ranged in thickness from 6 to 7 in. Those with nonuniform cross sections were of thickened-center design with thicknesses of 6-7-6 or 5-7-5 in. It was felt that the greater center thickness would prevent longitudinal cracking (22). Prior to World War I most concrete highway pavements were constructed to a uniform thickness of 4 to 6 in.

The thickened-edge design was developed in 1920 as a means of strengthening pavement edges. This type of cross section proved superior to comparable cross sections of uniform thickness in the relatively narrow pavements tested in the Bates Road Test (1920-1923) and the Pittsburg, California, Road Test (1921-1922). As a result it was quickly adopted and by 1934 41 states were using some form of thickened-edge design (21). The thicknesses of these nonuniform sections were usually 7-6-7, 8-6-8, and 9-7-9 or 9-6-9 in. As the use of wider pavements became more prevalent, some dissatisfaction with the section developed. By 1945 the states were about evenly divided between uniform-thickness and thickened-edge designs. Since 1956, all states with concrete pavement specifications have required uniform cross sections (21). Thickened-edge pavements are used in airfield pavement design to take care of edge stresses.

Pavement Type

Concrete pavement is generally classified as either plain, reinforced, or continuously reinforced. The first plain pavement (in Bellefontaine) has already been mentioned (6). One of the first reinforced pavements was constructed in 1908. This road was 24 ft wide, 11 miles long, contained superelevated lanes, and was placed in two courses to a total thickness of 5 in. (7). Reinforcement was recommended in 1914 to counteract cracking caused by thermally induced expansion and contraction (23). In 1916 it was recommended that all concrete roads be reinforced and specifications were written to cover several important design problems (24). In 1931 the common pavement slab in use in many states was of the thickened-edge design containing 30 to 60 lb of steel, wire mesh or bar mat per 100 sq ft (25).

The concept of continuous reinforcement was first tried experimentally by Indiana in 1938. This project led to the construction of test sections in Illinois in 1947, Texas in 1949, California in 1949, and New Jersey in 1947.

Jointing Practice

Early pavements were constructed without joints, but it soon became evident that it was necessary to introduce transverse joints to control transverse cracking. In 1914 the American Concrete Institute recommended that transverse joints should be not less than $^1/_4$ in. nor more than $^3/_8$ in. wide and should be placed across the pavement perpendicular to the centerline and not more than 35 ft apart. Randomized joint spacing is used in California (13, 19, 18, 12 ft) to avoid resonant response from vehicles.

As early as 1914 Illinois tried skewed joints with a skew angle of 60 degrees with the centerline of the roadway. Adjacent joints were skewed in opposite directions to make the irregularities less noticeable and to reduce cumulative vibrations (26). California constructed an experimental section of pavement with skewed joints in 1932. Skewed joints are now standard in California, Idaho, Colorado, and several other states.

Following World War I the use of steel dowels and proprietary devices to provide load transfer increased (27). Weakened plane contraction joints in which load transfer is achieved by aggregate interlock were introduced in 1919 (28) and have gained considerable acceptance through the years. Aggregate interlock has been the major means of load transfer for the closely spaced contraction joints of plain concrete. Mechanical load transfer devices are used universally in the joints separating the relatively long slabs of reinforced concrete pavements (21).

Early concrete pavements were constructed without longitudinal joints. As wider slabs were constructed and traffic increased, meandering longitudinal cracks developed near the centerline. Some engineers contended, even before 1914, that a pavement over 12 ft wide should have a longitudinal joint down the middle. They contended that dividing the pavement into two sections would serve to reduce warping stresses and to control longitudinal cracking, and would be more economical than building "thickened-center slabs" (26). The effectiveness of such joints in preventing longitudinal cracks in both plain and reinforced pavements was indicated by the Pittsburg, California, Road Test (12). During the 1920's the use of longitudinal contraction joints was generally adopted.

The occurrence of blowups in early pavements that had been in service for five or more years was considered evidence that concrete pavements required expansion joints to protect them from compressive stresses. By 1934 expansion joints were in general use, but with considerable variation in joint spacing among the state highway departments.

In 1934 the BPR required expansion joints at not more than 100-ft spacing and contraction joints at intervals of not more than 30 ft on all federal-aid road construction. Largely because of troubles experienced with construction and maintenance, some state engineers expressed a desire to increase the spacing of expansion joints or to omit them altogether (29). Based on experimental projects in 1940 where expansion joints were spaced from 100 ft to one mile apart, it was concluded that pavement design with contraction joints only could be adopted without fear of blowups. A great variety of expansion and contraction joint arrangements were used by the states in the period immediately after World War II. Since then the use of expansion joints has progressively declined except at structures and abutments. A study of pavement blowups made in Indiana was reported in 1945 (116). On the basis of this report the State of Indiana abandoned the use of expansion joints in concrete pavements except at ends of structures. A number of states followed this procedure within the next few years.

THEORIES FOR CONCRETE PAVEMENT DESIGN

Load Stresses

Engineers often speak of the theories of pavement design. In reality the numerous so-called theories all spring from a single theory, the theory of elasticity. A brief review of the pertinent aspects of the problem will be helpful in understanding subsequent work.

A pavement slab is variously called a slab-on-foundation, a pavement slab, a slab, or a plate-on-foundation. Regardless of the name applied, a pavement slab can be considered to be a plate with various support conditions. Timoshenko (30) distinguishes three kinds of plate bendings: (a) thin plates with small deflections, (b) thin plates wit large deflections, and (c) thick plates. Since the deflections of pavement slabs are small in comparison with their thickness, a satisfactory approximate theory of bendin of slabs by lateral loads can be developed by assuming that (a) there is no deformation in the middle plane of the slab (this plane remains neutral during bending), (b) planes of the slab initially lying normal to the middle plane of the slab remain normal after bending, (c) the normal stresses in the direction transverse to the slab can be disregarded (this assumption is necessary in the analysis of bending of the plate as will be seen later; approximate corrections can be made to account for pressures directly under the transverse load). With these assumptions all components of stress can be expressed in terms of the deflected shape of the slab. This function must satisfy a linear partial differential equation which, together with the boundary conditions, completely defines deflection w. The solution of this differential equation gives all necessary information for calculating the stresses at any point in the plate.

The approximate theories which define the behavior of thin slabs become unreliable for slabs of considerable thickness, particularly in the vicinity of highly concentrated loads. In these cases thick plate theory must be applied which considers the problem of the slab as a three-dimensional problem of elasticity. The stress analysis of such cases is complex and, according to Timoshenko, the problem is completely solved for only a few particular cases. In some instances the necessary corrections to thin plate theory are introduced at the points of application of concentrated loads. This is desirable in pavement slabs and has been discussed by Westergaard (3).

Timoshenko (30) has derived a differential equation which describes the deflection surface of pavement slabs subjected to loads applied perpendicular to their surface. This equation can be stated as

$$\frac{\partial^2 M_x}{\partial X^2} + \frac{\partial^2 M_{yx}}{\partial X \partial Y} + \frac{\partial^2 M_y}{\partial Y^2} - \frac{\partial^2 M_{xy}}{\partial X \partial Y} = q - kw \qquad (1)$$

where M_x is the bending moment acting on an element of the plate in the x direction, M_y is the bending moment acting on an element of the plate in the y direction, M_{xy} is a twisting moment tending to rotate the element about the x-axis (clockwise positive), M_{yx} is a twisting moment tending to rotate the element about the y-axis, q is the applied lateral load, k is the support strength of the subgrade, and w is the deflection at any point.

By introducing the appropriate moment equations (31, 32) the expression obtains

$$D\left(\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4}\right) = q - kw \qquad (2)$$

According to Timoshenko (30) this equation was obtained by LaGrange in 1811. The history of this development is given in Todhunter and Pearson's "History of Elasticity." The solution is complex and significant strides were not made until the early 1920's.

Corner Formula

The equations developed in the early 1920's by Goldbeck and Older for approximatin the stresses in concrete pavement slabs are empirical in nature. They apply only unde grossly simplified conditions. The best known of these formulas is generally called th "corner formula" and is expressed as

$$\sigma_c = \frac{3P}{h^2} \tag{3}$$

where

σ_c = maximum tensile stress in pounds per square inch in a diagonal direction in the surface of the slab near a rectangular corner;
P = static load in pounds applied at a point at the corner; and
h = depth of the concrete slab in inches.

This formula was derived using the assumptions of point load applied at the extreme corner and no support from the subgrade. The fiber stresses in the surface of the slab are assumed to be uniform on any section at right angles to the corner bisector.

Strain measurements taken on the Bates Road Test in 1922-23 appear to confirm the corner formula. Obviously the assumption of point load and load applied at the extreme corner were not correct for the Bates test sections. It is interesting to note that in spite of this there was reasonably good comparison. This good agreement could be partly due to the high impact transmitted to the slabs with the solid rubber tires used in the Bates test or to the fact that subgrade support may have been very low as assumed by this formula.

Westergaard Solutions

In 1926 Westergaard completed a solution of Eq. 2 and hence for the stresses in concrete pavement slabs. This analysis is concerned with the determination of maximum stresses in slabs of uniform thickness resulting from three separate conditions of loading: (a) load applied near the corner of a large rectangular slab (corner load); (b) load applied near the edge of a slab but at a considerable distance from any corner (edge load); and (c) load applied at the interior of a large slab at a considerable distance from any edge (interior load). In his solution of this problem, Westergaard made the following important assumptions:

1. The concrete slab acts as a homogeneous isotropic elastic solid in equilibrium.
2. The reactions of the subgrade are vertical only and they are proportional to the deflections of the slab.
3. The reaction of the subgrade per unit of area at any given point is equal to a constant k multiplied by the deflection at the point. The constant k is termed "the modulus of subgrade reaction" or "subgrade modulus," and is assumed to be constant at each point, independent of the deflection, and to be the same at all points within the area of consideration.
4. The thickness of the slab is assumed to be uniform.
5. The load at the interior and at the corner of the slab is distributed uniformly over a circular area of contact; for the corner loading the circumference of this circular area is tangent to the edge of the slab.
6. The load at the edge of the slab is distributed uniformly over a semicircular area of contact, the diameter of the semicircle being at the edge of the slab.

For the three cases given and the appropriate assumptions as listed, the following expressions for stress were developed by Westergaard:

$$\sigma_i = 0.31625 \frac{P}{h^2} \left[4 \log_{10}\left(\frac{\ell}{b}\right) + 1.0693\right] \tag{4}$$

$$\sigma_e = 0.57185 \frac{P}{h^2} \left[4 \log_{10}\left(\frac{\ell}{b}\right) + 0.3593\right] \tag{5}$$

$$\sigma_c = \frac{3P}{h^2} \left[1 - \left(\frac{a\sqrt{2}}{\ell}\right)^{0.6}\right] \tag{6}$$

where

P = point load, in pounds;

σ_1 = maximum tensile stress in pounds per square inch at the bottom of the slab directly under the load, when the load is applied at a point in the interior of the slab at a considerable distance from the edges;

σ_e = maximum tensile stress in pounds per square inch at the bottom of the slab directly under the load at the edge, and in a direction parallel to the edge;

σ_c = maximum tensile stress in pounds per square inch at the top of the slab, in a direction parallel to the bisector of the corner angle, due to a load applied at the corner;

h = thickness of the concrete slab in inches;

μ = Poisson's ratio for concrete (taken as 0.15 in these equations);

E = modulus of elasticity of the concrete in pounds per square inch;

k = subgrade modulus in pounds per cubic inch;

a = radius of area of load contact in inches (the area is circular in the case of corner and interior loads and semicircular for edge loads);

b = radius of equivalent distribution of pressure at the bottom of the slab = $\sqrt{1.2\,a^2 + h^2} - 0.675\,h$; and

ℓ = the radius of relative stiffness = $\sqrt[4]{\dfrac{Eh^3}{12\,(1 - \mu^2)\,k}}$.

Modifications to the original 1926 equations were made by Westergaard in 1933, 1939 and 1947. The 1933 modifications were concerned primarily with interior loads and will not be discussed here.

Slab on Elastic Solid

Other major theoretical work applicable to concrete pavement design involves the solution of large slabs supported on an elastic solid. Work in this area has been done by Hogg (33) and Holl (34) among others. More complicated loading cases have been considered by Volterra (35) and Bergstrom (36).

The case of a large slab supported by an elastic solid layer of finite thickness is perhaps more realistic than the semi-infinite solid case. Pickett (37), Holl (38), and Burmister (39) have considered this case among others. The mathematics of these solutions are quite complex and some assumptions are necessary to make solution possible.

Pickett et al (40) contributed greatly to the solution of these slab-on-elastic-solid problems with their work at Kansas State College, published in 1951. They considered the equations governing the slab on an elastic foundation. Where possible they solved these equations in closed form; where solutions were not obtainable they developed numerical techniques for evaluating the equations. They further developed influence charts to make the results readily usable by practicing engineers. They also extended the methods to rectangular slabs supported on elastic solid layers of finite thickness.

Layered System Analyses

Some authorities consider that pavement behavior is more closely approximated as a three-layered system than as a slab-on-foundation problem. Classical work in the analysis of a two-layered system has been done by Burmister (41) and by others. The analyses of three-layered systems are much more complex than single- or two-layered systems. Because of the great number of variables involved, solutions have only been made for surface deflections between the center of the circular, uniform, vertical load and for certain stresses and strains beneath the same load. These structures cover a wide range of values of the physical constants which are applicable to ordinary pavement structures. Burmister (42) laid much of the groundwork for the solution of two elastic layers on a semi-infinite elastic subgrade. He did not, however, provide numerical evaluation of the deflection or stress. Evaluation of these stresses has been accomplished by Hank and Scrivner (43) and Peattie and Jones (44) among others. To date these equations have been used primarily in the evaluation and analysis of flexible pavements. In reality they are probably more applicable and can be more helpful in the design of concrete pavements than in the so-called flexible pavements. At the present time, however, they await further evaluation and application.

Analysis of Finite Pavement Slabs With Discontinuities and Nonuniform Pavement Support

The theories described previously involve single-load application, and the solution of the equations imposes limiting assumptions on realistic pavement problems. All the solutions involve uniform slab thickness, uniform homogeneous isotropic (or special-case orthotropic) slabs, uniform foundation support, and certain uniform or special-case loading conditions. Lateral loads are considered, but in-plane forces and applied couples or moments cannot be handled.

Hudson and Matlock (32) have developed a method for analyzing slabs which is not limited by many of these previous limitations. The method involves the formulation of the problem by finite element techniques and the solution of the resulting equations by numerical methods in a large digital computer. The general method is suitable for analyzing orthotropic and isotropic slabs and is not limited by discontinuities. The method allows considerable freedom in plan configuration loading, flexural stiffness and boundary conditions. Three principal features are incorporated into the method: (a) the plate is defined by a finite element model, and the components of this model are grouped for analysis into orthogonal systems of beam-column elements and forces; (b) each individual line-element of the two-dimensional system is solved rapidly and directly by recursive techniques; and (c) an alternating-direction iterative method is utilized for coordinating the solution of the individual line-elements into the slab solution. The method necessitates the use of high-speed digital computational equipment, but programs for the solution have been developed and debugged and are available for solution of all classes of problems.

Other Work

In the 1930's, F. T. Sheets introduced an equation containing a constant, c, which was equated to the value of k as employed by Westergaard. The Sheets equation can be written as

$$\sigma_c = \frac{2.4 \; P \; (c)}{h^2} \qquad (7)$$

This equation is reported to give stresses which are in good agreement with those obtained at the Bates Road Test. However, this equation is no longer in general use and does not contain all the variables of interest to the designer.

Many subsequent stress equations are based on some modification of the original Westergaard equation. The major work which resulted in these modifications included the Kelly equation developed as a result of the BPR Arlington Test, the Spangler equation developed as the result of the Iowa State College tests and the Pickett equations developed as the result of additional mathematical analysis.

Special Theories Applicable to Particular Pavement Types

The first portion of this section described the various load-stress equations developed for use in concrete pavement design. These equations have general applicability for determining pavement thicknesses required to resist certain traffic loads. In the design of particular pavement types, however, certain of these equations are used more than others. In addition, special theories involving the design of reinforcement and prestressing are sometimes involved. This part discusses the application of load-stress theories and other special theories which are needed for the individual pavement types.

Plain Concrete Pavements—Plain concrete pavements are ordinarily designed utilizing one of the basic theories described earlier. Probably a majority of the pavements have been designed using Westergaard's corner loading formula with or without subgrade support under the corner. In some cases a completely unsupported corner with no load transfer is utilized, but in some cases partial transfer of the load to an adjacent slab through some type of load transfer is considered, thus reducing required pavement thickness. Plain pavements require no other special theories in design.

Jointed Reinforced Concrete Pavement—Thickness design of jointed reinforced concrete pavement is basically the same as that for plain concrete pavement with the exception that some designers use Westergaard's edge loading formula for highway design and his modified edge loading formula for airport design. Those who use corner loads always consider some load transfer since common design dictates mechanical load transfer devices between slabs of reinforced pavement.

Reinforcement design for rigid pavement is based on the concept that since it is often not economically possible to prevent the formation of cracks, it is necessary to control the opening of cracks in such a manner that the original load-carrying capacity of the slab is preserved. If the crack is permitted to open, contact between the faces of the crack is lost, with a corresponding loss in shearing resistance, and continued application of load results in progressive breakage. The function of the steel reinforcement is to hold the interlocking faces of the crack in tight contact and thus render the crack shear-resistant. As a consequence, load transfer is maintained across the crack and both slab ends act together as a load approaches. This design is based on the concept that this action maintains the structural integrity of the slab and prevents excessive deflections.

Since the principal function of steel reinforcement in rigid pavement is to hold the interlocking faces of the concrete at a crack in tight contact, it is only necessary to furnish sufficient steel area to resist the forces tending to pull the crack faces apart. These forces develop when the slab tends to shorten as a result of a drop in temperature, concrete shrinkage, and/or a reduction in moisture content. As the slab contracts, the movements are resisted by the friction between the slab and the underlying subgrade or subbase. The resistance to movement produces a direct tensile stress and may cause the concrete to crack. As soon as the concrete cracks, the tensile stress is transferred to the steel reinforcement.

Since the crack can occur at any location in the slab, the steel must be designed to resist the maximum frictional force that can be developed in any one section of the slab. This force is considered to be a maximum, for a slab of given length, at a distance from the nearest free edge equal to half the slab length. Using the notation given below, the maximum force is equal to $L/2 \times Fw$ pounds per foot of width. The tensile resistance of the reinforcing steel per foot of width is equal to $A_s f_s$. Equating these external and internal forces gives

$$A_s = \frac{FLw}{2f_s} \tag{8}$$

where

A_s = required area of steel per foot of width or length, sq in.;
F = coefficient of subgrade friction;
L = distance between free joints, ft;
w = weight of slab, lb per sq ft; and
f_s = allowable tensile stress in steel, psi.

Continuously Reinforced Concrete Pavement—Continuously reinforced concrete pavement is a relatively new concept of pavement design in that transverse joints long considered essential in the construction of concrete pavement are eliminated. In their place the pavement is allowed to crack in a seemingly uncontrolled random pattern and these cracks are held tightly together by continuous steel reinforcement.

The same theory for thickness design can be used for continuous pavements as for jointed pavements, if it is first considered that the concrete and steel are going to act separately. The purpose of the concrete is to carry the wheel load, and the purpose of the steel is to keep cracks tightly closed so that there is effective load transfer across these cracks. If the cracks are kept tightly closed so that no water will seep through them to harm the subgrade, then pumping and other detrimental effects are reduced.

By removing the joints in the pavement it is theoretically possible to design continuous pavements based on interior loads or edge loads. Ledbetter and McCullough (46) show that using the interior load design results in a 20 percent savings in concrete thickness over the regular or jointed pavement designed for corner loading. The same equations cited for use with jointed pavements can be used for continuous pavements by making proper choice of loading conditions.

Design of Steel Reinforcement—The purpose of the continuous steel reinforcement is not to prevent cracking. Its purpose is to hold the cracks which do result tightly closed and provide practical continuity of slab across the crack. The design of this steel is often based on the basic relationship proposed in 1933 by Vetter (47). This relationship states that the percent of steel required to control volume change is equal to the tensile strength of the concrete divided by the tensile strength of the steel. This approach is discussed thoroughly by Ledbetter and McCullough (46). It is appropriate to point out that this relationship indicates that stronger concrete requires more steel in order to be adequately designed, that is, in order to resist contraction of the concrete.

To illustrate the theory involved here, assume a temperature drop which causes the concrete to contract. Restrained contraction induces tensile stresses in the steel at cracks and builds up tensile stresses in the concrete between cracks until the tensile strength of the concrete is reached. At this point the concrete cracks again, thus reducing the stress in the steel at the existing crack. With this concept, less steel than heretofore considered necessary is needed because only enough steel is required to insure that the concrete tensile strength is reached before the steel stress at the crack reaches the yield point or some percent of the yield point.

In the 1962 AASHO Interim Guide for the Design of Rigid Pavement Structures (111) the Committee on Design recommended that the percentage of longitudinal steel in a CRC pavement be determined by correlating a number of variables (such as tensile strength of the concrete, yield strength of the steel, frictional resistance of the subbase) and by using engineering judgment based on experience. In general, the theoretical minimum percentage of longitudinal steel (P_s) may be determined by

$$P_s = (1.3 - 0.2F) \frac{S_t}{f_s - N S_t} \times 100 \tag{9}$$

where

P_s = ratio of area of longitudinal steel to area of concrete, percent;

F = coefficient of friction between pavement and subbase;

S_t = tensile strength of concrete, psi (about $0.4\ S_c$, modulus of rupture);

f_s = allowable working stress in steel, psi;

N = E_s/E_c;

E_c = modulus of elasticity of concrete, psi; and

E_s = modulus of elasticity of steel, psi.

The formula is based on the assumptions (a) that sufficient bond area is provided to develop the full working stress of the steel, and (b) that adequate load transfer is provided at transverse construction joints. The frictional factor, F, depends upon the surface smoothness of the subbase immediately beneath the rigid pavement. The value of F may range between 1 and 2, with 1.5 commonly used. In general, the friction factor may be set according to the following standards:

F = 1.0 for smooth-textured subbase surface,
F = 1.5 for medium-textured subbase surface, or
F = 2.0 for rough-textured subbase surface.

Equation 9 will generally govern the percentage of longitudinal steel required in a continuously reinforced pavement. Under severe temperature variations or when unusual material properties are encountered, a formula which considers the thermal coefficient may be required:

$$P_s = (1.3 - 0.2F) \frac{S_t}{2\,(f_s - T\epsilon E_s)} \times 100 \qquad (10)$$

where T = temperature range in degrees F, and ϵ = thermal coefficient of concrete and steel. The percentage of steel should be calculated by both formulas, under the special conditions outlined, and the highest value used in design.

Prestressed Concrete Pavements—Prestressed concrete pavements are more complex to design than conventional pavements. Design problems are increased because prestressed pavements contain many features not used in other pavement types. For example, special devices are needed to prestress the concrete; friction-reducing layers are necessary to reduce subgrade restraint stresses in relatively longer slabs; special transverse joints may be needed to allow large horizontal movements with controlled vertical movements; and end abutments may be required. Other methods for determining load stresses and deflections are not adequate for design of prestressed pavements. In the design of the other pavement types, it is assumed that load stresses and deflections remain within the elastic range of the concrete and therefore design methods based on elastic theory can be used. Prestressed concrete pavement studies (48, 49, 50) concerned with load response indicated that prestressed pavements can adequately support interior and edge loads of greater magnitude than those causing bottom surface cracks. These bottom cracks serve as partial plastic hinges under passage of load. As the load is removed, the cracks are closed by the prestressing forces. In the design of prestressed pavements it is thus necessary to determine stresses and deflections after bottom surface cracking or for conditions not covered by elastic theory. Theoretical methods for determining these values have been reported for the case of interior loading by Levi (51), Cot and Becker (52), and Osawa (53). These analyses are limited to a concrete slab supported by a dense liquid subgrade and prestressed equally in longitudinal and transverse directions. The principal difference between the method used by Levi and the methods used by Cot and Becker and Osawa is the type of cracking assumed at the bottom surface of the slab. Levi assumed an initial circular bottom surface crack to simplify the computation of moments from slopes by use of the reciprocal theorem. In the methods used by Cot and Becker and Osawa, it was assumed that bottom surface cracking occurred in a radial pattern. The radial crack assumption seems more reasonable because the elastic theory shows the tangential moment to be greater than the radial moment near the loading point. The Cot and Becker solution selects a length for the bottom surface radial crack and determines the load required to produce a crack of this length. In contrast, the Osawa solution is a step-by-step procedure whereby an increment of load results in an incremental increase in the length of the bottom surface radial cracks. Each of these theoretical solutions provides a method for predicting the load causing top surface circular cracking. Top cracking occurs at a load of more than twice that causing bottom cracking in most prestressed pavements and is considered to be an indication of pavement failure.

In prestressed pavement design, consideration must also be given to stresses other than those caused by wheel loads. A method (54) based on both theory and results of pavement research has been proposed for computing subgrade restraint and temperature warping stresses in 400- to 800-ft long prestressed slabs. These stresses were combined with wheel load stresses to prepare recommendations for required amounts of longitudinal and transverse prestress in highway pavements.

Because prestressing a concrete pavement increases its load capacity, the thickness of a prestressed pavement may be less than other rigid pavement types for support of equal loads. The loss of all prestress at any section of a prestressed pavement could result in a failure caused by insufficient thickness to support design traffic loads. Therefore, it is essential that a certain minimum amount of prestress be maintained at all times and at all locations of a prestressed pavement. To assure that prestress is maintained, some additional prestress should be applied initially to compensate for certain losses that will occur during and following construction. These losses could result from elastic shortening, creep, shrinkage of the concrete, relaxation in the steel, anchorage losses, tendon friction in post-tensioned systems, and hygrothermal contraction. Theoretical methods for determining these losses have been developed for most prestressed members. Accepted procedures for determining such losses in prestressed pavements have been summarized elsewhere (55).

ACCOMPLISHED RESEARCH AND SERVICE EXPERIENCE

Since the first appearance of concrete pavements there has been a continuous effort to improve design. This research effort includes (a) laboratory experimentation, (b) theoretical analysis, (c) performance studies of existing pavements under actual service conditions, (d) observations and tests on specially constructed pavements for a long period of years under normal traffic, (e) accelerated testing of selected sections of existing pavements and specially constructed pavements under controlled traffic, and (f) accelerated testing with specially built load carts on prototype pavements.

Observations of pavement performance under actual service conditions and normal traffic comprise many variables that are difficult to analyze and correlate with other factors under study. These observations must often continue for several years before the results can be evaluated.

All rigid pavement research emphasizes the interrelationship of traffic loading, environment and subgrade support. Each plays its part in prolonging or reducing the service life of the pavement and each has been the subject of much study and experimentation.

Traffic Loading

Magnitude of wheel loadings, wheel distribution and high airplane tire pressure and the number of load repetitions are the most important factors that influence the useful life of a properly designed and constructed pavement. Although this statement is self-evident, design criteria combining these factors could not be established until research from experimental tests became available.

Early Road Test—The Bates Road Test demonstrated how an increase in the magnitude of the wheel load (dual solid rubber tires) and an increase in the load applications reduced the service life of pavements varying in type and thickness. It produced the first rigid pavement design critéria based on wheel load and allowable stress in the concrete.

In order to obtain up-to-date information on the relative effects of load repetitions corresponding to the large volumes of heavy trucks using the highways after World War II, the controlled traffic test, Road Test One-MD (16), was conducted on a section of an existing highway in Maryland. This test established that a 32-kip tandem axle caused greater distress than an 18-kip single axle; similarly a 44.8-kip tandem-axle load was more severe than a 22.4-kip single-axle load.

Data from Road Test One-MD showed that stresses and deflections measured at vehicle speeds of 40 mph were about 20 percent less than those at creep speed. It was found that wheel placement at 30 in. from the pavement edge gave edge stresses and deflections about 50 percent less than those recorded with the wheel 6 in. from the edge.

AASHO Road Test—The most comprehensive accelerated test under controlled traffic to date is the AASHO Road Test (56) at Ottawa, Illinois (1958-1961). The test pavements were subjected to over one million repetitions of single-axle loads ranging from 2 to 30 kips or tandem-axle loads from 24 to 58 kips. Analysis of the results produced rigid pavement performance equations which make it possible to determine the tandem-axle load in terms of equivalent single-axle load. These performance equations have been extended (57, 111) to cover the conversion of mixed traffic into equivalent single-axle loads. Equivalence factors have been developed which make it possible to estimate the effects of different axle loadings on the pavement structure. Since these equations include the effect of load repetitions, they furnish a basis for estimating the design requirements for a pavement to remain serviceable for a given number of years under the anticipated traffic as determined by traffic surveys. Also, they provide a basis for estimating the service life of existing pavements under fixed or changing traffic conditions.

U.S. Corps of Engineers Tests—During the period 1941 to 1956 the Corps of Engineers conducted for the Air Force surveys of existing pavements and accelerated testing with controlled traffic on existing pavements and on specially constructed prototype pavements (58, 59, 60, 61, 62). Some basic findings with respect to traffic loadings discussed by Mellinger, Sale and Wathen (62) are:

1. The repetitive traffic of slow-moving aircraft is the most severe loading to which airfield pavements are subjected; this occurs on taxiways, runway ends and aprons.
2. Impact on normal landings can be ignored due to lift on the wing surfaces at aircraft landing speeds.
3. Early studies indicated that 5000 coverages were representative of 10 to 20 years of pavement life.
4. Later studies showed that the heavier aircraft tended to confine their travel to the central portion of a taxiway, resulting in channelized traffic indicating that 30,000 coverages could be expected in 10 to 20 years of pavement life.

The term coverage is defined as a sufficient number of vehicle operations to produce one application of the design load over the entire width of the traffic area. In the tests described, a coverage was equivalent to three repetitions of the wheel loading (62).

Theories give concrete stresses on the basis of one application of the load. Therefore a design factor must be added to compensate for fatigue of the concrete due to load repetitions as well as other factors to compensate for the limited efficiency of load transfer devices at joints, temperature stresses, impact and eccentric loading (particularly in the case of loading by airplane landing gears). The design factor being principally related to the life expectancy of the pavement (the Corps of Engineers uses 25 years for roadways with normal maintenance) makes it possible for the designer to relate pavement stresses of various magnitudes to an equivalent number of applications of a constant stress.

The Corps of Engineers developed a method of correlating traffic volume and load intensity with equivalent 18,000-lb single-axle dual wheel load operations (119, 150). Factors can be developed based on statistical analysis of the relationship of operations to width of pavement, width of tire contact area, number of wheels on the axle, spacing of wheels and degree of wander to produce one coverage. Airfield pavement data are available to 30,000 coverages and can be extended on the basis of the fatigue characteristics of concrete for greater vehicular traffic. This establishes the coverage design factor as it affects the design thickness obtained from stress consideration.

Using the Westergaard analysis for loading tangent to a free edge and incorporating a dynamic factor of 1.55, the Corps of Engineers would require a thickness h for 5000 coverages:

$$h = \left[\frac{6P}{\sigma}\left(1.55\,\frac{M}{P}\right)\right]^{1/2}$$

where P is the wheel load, σ is the design stress in the pavement, M/P is the maximum moment per pound of wheel load induced by all wheels on the axle, which is a function of A/ℓ_2 (A is the tire contact area and ℓ the radius of relative stiffness) for a given wheel spacing. During the anticipated 25-year life, the number of coverages will be a factor of the assumed 5000, and likewise the thickness will be a related percent of h ranging from 82 percent for 1 to 158 percent for 300,000,000 equivalent coverages (from the minimum of the low range to the maximum of the high range).

Environment

Pavement performance, like human behavior, is influenced by environment. Environmental factors such as frost, rainfall, and temperature, although variable in intensity from year to year, are constantly at work. Thus it is necessary to adjust designs accordingly. Statistical data and general information on frost, rainfall and temperature are available in publications of the Department of Agriculture (63) and the U.S. Weather Bureau.

Frost—Although the effects of frost action on pavements cannot be separated from temperature and rainfall, it is the most severe of the environmental factors and has been the subject of a large volume of research in the laboratory and in the field (64, 65, 66, 67, 68). Formulas have been developed for predicting the depth of frost penetration, and the Corps of Engineers (69) has produced empirical curves giving the relationship between frost penetration and freezing index.

Rainfall—The influence of rainfall on the stability and strength of the supporting medium (subgrade and subbase) has long been recognized, and rainfall records are normally included in important pavement research. Although it has not been possible to make a quantitative determination of the effects of rainfall, it is well established as a factor in (a) the moisture content of the subgrade and subbase (strength and volume change), (b) the elevation of the water table, (c) the intensity of frost action, (d) erosion, (e) pumping, and (f) infiltration.

The moisture content of the concrete slab will vary with rainfall and will affect the expansion and contraction.

Temperature—Air temperature has a direct influence on both the rigid slab and the supporting medium. Design of the horizontal slab dimensions is controlled by the permissible change in length and width resulting from changes in air temperature. These changes also control the design of steel reinforcement.

Variations in temperature between the top and bottom of the rigid slab affect warping and curling and, consequently, deflections.

Intensity of frost action is directly related to air temperature and will influence the subbase design.

Structural Variables

Subgrade—Ever since 1920 there has been a continuing effort to advance our knowledge of subgrade soils and the application of this knowledge to the design of pavements. Standard methods of surveying, sampling and testing of soils have been adopted by AASHO (70). Three soil classifications based on these standards, or modifications of them, have been established. In general, the highway departments and the Bureau of Public Roads use the AASHO classification which is based primarily on grain size, liquid limit and plasticity index. The Federal Aviation Agency, using the same tests and methods, has set up a classification (71) that is applied to pavement design on civil airports. The Unified System, also based on grain size and plasticity tests, is used by the Corps of Engineers (72), the Department of the Air Force and the Bureau of Yards and Docks (73) for military pavements.

These classifications, which combine a knowledge of the physical properties of the soil as determined by laboratory tests and their performance as subgrades under pavements, serve to identify a subgrade soil as having physical properties similar to those of known behavior. Therefore, they can be expected to furnish the same degree of stability under the same conditions of moisture and climate.

The soil profile, determined by borings, shows the arrangement of the different so layers, the elevation of the water table and the other properties that supply informati on the need for subsurface drainage and its design.

With a knowledge of environmental factors such as topography and climate the engi neer can utilize the information from the subgrade survey and the classification tests estimate the susceptibility of the soil to volume change, frost action and pumping. Als these data disclose the existence of conditions of nonuniform support, detect resilient soils that are subject to detrimental deflection and rebound, and indicate the feasibilit of improving the stability of the soil by means of additives.

There are a number of other tests (74) that can be performed in place and in the laboratory to estimate the strength and supporting power of the soil. The most widely used test for rigid pavements is the plate loading test to determine the modulus of sub grade reaction referred to as the k value. As a result of research (75) the 30-inch diameter plate has been most widely adopted for the test with a given loading procedu keyed to the results.

The subject of expansive subgrade soils has been the basis for many investigations A bibliography on this subject is provided in reference numbers 123 through 149 inclusiv

Subbase—Some years ago, rigid pavements designed strictly on the basis of the weight distribution and repetition of loads were considered adequate to carry any traff regardless of the type of subgrade soil on which they were placed. With the rapid increase in the volume of heavy vehicles it became evident that the service life of the pavement was greatly affected by the stability and strength of the subgrade soil. As a result, practically all heavy-duty rigid highway pavements and a few airport pavemen include a subbase as part of the structure. The subbase may consist of one or more layers of granular or stabilized material, properly compacted, between the subgrade and the rigid slab. This subbase may serve one or more of the following purposes: (to provide uniform support; (b) to increase the supporting power above that provided b the subgrade soil; (c) to minimize the detrimental effects produced by volume changes in the subgrade; (d) to minimize or eliminate the detrimental effects of frost action; a (e) to prevent pumping.

Granular materials, dense-graded and open-graded, are most commonly used to ac complish these purposes. In many cases they are available in such quality that the on requirements are uniform distribution and good compaction. Marginal materials may be improved by stabilization to meet specific requirements.

Field and laboratory investigations (56, 76, 77, 78, 79) have shown the effectivene of granular materials in overcoming subgrade problems, provided they have the prope physical properties, are adequate in thickness, and properly compacted. Highway departments and airport authorities have adopted standard specifications (70, 73, 80, 81 82) covering composition, plasticity index and compaction.

Thickness requirements are determined largely by experience. Four- to nine-inch thicknesses are common for the purpose of providing uniformity of support and strengt Greater thicknesses, 12 in. or more, may be required on highly active clays, very un stable soils in poorly drained locations, and highly resilient soils. Some wheel loads encountered on airports may also require thicknesses greater than 12 in.

Subbase design for frost action and pumping are special situations and are covered in the discussions that follow.

Frost Action—Frost action in relation to rigid pavements refers to the heaving whic takes place when the ground freezes and loss of stability when thawing occurs. The mechanics of frost heave, the loss of strength under thawing conditions and methods o prevention have been rather well covered in the literature (64, 65, 66, 67, 68). Ther is general agreement that the occurrence and magnitude of detrimental frost action depends on the character of the soil, the depth and rate of freezing, and the availability of water. Frost heave studies have disclosed that very fine sands, silts, and some clays are the worst offenders, while well-drained sandy and gravelly soils are not sub ject to heaving. Serious differential heaving was observed where soils such as silts and very fine sands were found as pockets surrounded by well-drained sandy soils.

Heaves were encountered also in sandy and gravelly materials where they were maintained in a saturated condition by blocked drainage or a water table close to the surface.

Detrimental frost action can be reduced in most granular materials if underdrains are installed at proper locations so as to intercept and remove water that would otherwise tend to collect in them.

Other frost-susceptible soils are sometimes excavated to a specified depth and replaced with non-frost-susceptible material or are blanketed with a layer of this material. Among highway departments the thickness of selected material ranges from one-half to the full depth of frost penetration—mostly from one-half to two-thirds.

More interest is being shown among highway departments in the use of freezing index data to determine depth of freezing and required thickness of selected material. This method, used by the Departments of the Army and Air Force and the Navy Bureau of Yards and Docks, is described in the pavement design manuals of these agencies (69, 73) as well as in a paper by Linell, Hennion and Lobacz (67).

Pumping—The problem of rigid pavement pumping took on such proportions in 1942 that the publication of "Wartime Road Problem No. 4, Maintenance Methods for Preventing and Correcting the Pumping Action of Concrete Pavement Slabs" was published in October of that year. The HRB Committee responsible for this report sponsored further studies and issued a final report (83) in 1948. This report defined pumping as the "ejection of water and subgrade soil through joints, cracks and along the edges of pavements caused by downward slab movement actuated by the passage of heavy axle loads over the pavement after the accumulation of free water on or in the subgrade." In 1957 Yoder (84) reported the results of a study of pumping of highway and airport pavements. These reports and others (76, 77, 78, 79) agree that three basic conditions must be present to create pumping. They are (a) frequent heavy loads, (b) fine-grained soils that will go into suspension with water, and (c) free water under the pavement.

Early investigators agreed that in many instances pumping was eliminated when granular subbases were included as a part of the rigid pavement structure. However, Yoder (84) points out that large volumes of heavy trucks will cause pumping if the granular subbase has an excessive amount of particles passing the No. 200 sieve. He indicates that 10 percent passing the No. 200 sieve may be the limit under severe conditions. Pumping development on Road Test One-MD (16) supports these findings.

On the AASHO Road Test (56) all failures in rigid pavements were preceded by pumping of material from beneath the concrete slabs. Generally, this material consisted of subbase gravel including the coarser fractions. Pumping of embankment soil was generally confined to those sections constructed without subbase. Severe pumping of the subbase material was experienced only in sections with the two thinner slab thicknesses. However, some pumping appeared in all but one of the sections. Very little material pumped through joints or cracks. The major pumping was along the pavement edge. The importance of a subbase layer was reiterated at the AASHO Road Test even though there was no difference in performance for subbase thicknesses from 3 to 9 in. There was, however, a significant improvement in the use of a granular subbase as opposed to placing the pavement directly on clay.

It appears from all these studies that (a) the subbase should be well graded from coarse to fine and that the percentage passing the No. 200 sieve can vary depending on the maximum size of the graded material; and (b) in open-graded materials the minus No. 200 material should be kept to a minimum with a maximum of 10 percent while up to 15 percent may be permissible in dense-graded materials and up to 10 percent in sandy types.

Stabilization—When available granular materials for subbase do not conform to specification requirements, they can be improved to an acceptable standard by means of stabilization. Mechanical procedures or chemical additives may be used to accomplish this improvement.

Mechanical stabilization consists in the introduction of a second material in such proportions that the required gradation and physical requirements of the specifications are met.

It may be more economical in a particular area to use an admixture of portland cement, lime or bituminous material to improve substandard or marginal materials. Both lime and portland cement have been widely used to reduce plasticity and as a binder for a great variety of soils and soil aggregate mixtures. Bituminous materials have been used as a binder to produce effective stabilization of some marginal subbase materials.

When granular materials must be hauled long distances, it has been found economical to reduce the subbase thickness and stabilize the subgrade soil. This problem generally arises in areas where the predominant soils are the plastic types, and subgrade stabilization under these conditions is becoming more popular. Portland cement, lime, calcium chloride and combinations of portland cement and lime, mixed with the soil and compacted to a depth of 6 inches, are used for this purpose.

Stabilization is generally covered by highway department and federal specifications. Publications of the Portland Cement Association (85), the Asphalt Institute (86), and the National Lime Association (87) contain recommendations covering the details of testing, design, and construction. Publications of the Highway Research Board (88, 89, 90, 91, 92) contain additional information.

Compaction—Modern construction practice requires density and moisture control when compacting subgrades and subbase material. Ever since Proctor (93) brought out the relationship existing between the soil, the degree of compaction and the resulting density, a great deal of effort has been expended to develop criteria applicable to compaction control for different types of material (94, 95, 96). This has been especially important in connection with the compaction of granular materials for subbases, since vibratory action of traffic can result in differential settlement of the rigid slab if the subbase is not properly compacted. All constructing agencies such as the various highway departments represented by AASHO (70), the Corps of Engineers (80), the Bureau of Yards and Docks (73), the Federal Aviation Agency (81), and the Bureau of Public Roads (82) specify definite compaction requirements (density and moisture content).

Slab Dimensions and Characteristics

Thickness—As noted earlier, the corner formula for estimating the required thickness of a rigid pavement was proposed by Goldbeck (1) in 1919 and later applied by Older (2) in analyzing the results of the Bates Road Test in 1924. Prior to the Bates Road Test most pavement slabs were uniform in cross section and from 4 to 6 in. thick. As a result of the Bates Road Test the thickened-edge cross section was widely adopted and the prevailing thicknesses were 8-6-8 and 9-7-9 in.

In the early 1930's the Bureau of Public Roads conducted extensive investigations on the structural design of concrete pavements (13). Among many other important findings it was concluded that, so long as the basic conditions assumed for the analysis are approximated, the Westergaard theory describes quite accurately the action of the pavement. From tests carried on in the 1940's by the Corps of Engineers (58) a similar conclusion was drawn with respect to airfield pavements.

Accelerated tests under controlled traffic, such as Road Test One-MD (16) and the AASHO Road Test (56) for highway pavements and the Lockbourne (59, 60, 61) and Sharonville (62) tests for airfield pavements, produced information on the influence of dynamic load repetitions on slab thickness requirements. As a result of these investigations, and the fact that higher speeds on highways required wider pavements with the consequence that the wheels of vehicles could operate at a greater distance from the pavement edge, a cross section of uniform thickness has been generally adopted.

With the development of continuously reinforced concrete pavements and the elimination of transverse joints (97) it appears that the thickness of a concrete pavement, continuously reinforced with adequate steel area, can be reduced.

Warping—Kelley (98) has pointed out that as early as 1926 Westergaard presented a theoretical analysis of warping stresses due to temperature but their importance was not generally recognized and it remained for the Arlington tests (13) to demonstrate their magnitude. Based on the theoretical analysis, Bradbury (99) developed general equations for the computation of temperature warping stresses. Using these equations, Kelley shows that the maximum warping stresses can go as high as 275 and 375 psi, respectively, for a 6- and 9-in. slab.

Data from Road Test One-MD (16) show that stresses and deflections caused by loads acting at the corners and edges of a pavement slab were influenced to a marked degree by temperature warping of the slab. For the corner loading the stresses and deflections for a severe downward-warped condition were observed to be only approximately one-third of those for the critical upward-warped condition. For the free-edge loading the effect of temperature warping, although appreciable, was not as pronounced as for the case of the corner loading.

An important observation quoted from the report of Road Test One-MD states, "It is noted that the values of the combined load and warping stresses greatly exceeded one-half of the modulus of rupture value of the concrete. At first thought, and in light of the fatigue properties of the concrete determined from laboratory tests, it would seem that such high stress values would soon cause failures in the structure. However, this was not the case of the sections in the experimental pavement, even after 10 years of service."

All studies (Arlington, One-MD and AASHO) show higher stresses when the slab is warped upward than when warped downward. The warped down condition gave a reduction in stresses on the AASHO Road Test, but the reduction was less on a percentage basis than that on Road Test One-MD.

A report on warping of concrete pavements was published in 1945 (45).

Jointing—In order to control certain stresses adequately, it is necessary to limit the width and length of slabs by means of joints spaced at suitable intervals. Most of the available information relating to joint spacing and joint design has been obtained from field inspections of pavements in service. As a result it is known that horizontal slab dimensions, which determine the spacing of joints, depend primarily on whether the pavement is reinforced or nonreinforced. Other factors may include environment, materials and thickness.

It has been found that the formation of random longitudinal cracks can be eliminated from highway pavements 5 to 10 in. thick if they are divided by longitudinal joints into lanes not over about 12 ft wide.

On airports, where greater thicknesses are common, the design manuals of the Federal Aviation Agency (71), the U.S. Navy (73), and the Corps of Engineers (100) permit the construction of pavement widths of 25 ft without a center joint when the pavement is over 10 in. thick. With respect to transverse contraction joints, nonreinforced slabs may be from 15 to 25 ft long depending on thickness while adequately reinforced slabs may be up to 75 ft long regardless of thickness. Nonreinforced slabs, generally, are limited to lengths of 15 to 20 ft for slabs up to 10 in. thick and from 20 to 25 ft for slabs over 10 in. thick. In practically all cases these are contraction joints. Randomized joint spacing is used by some agencies.

Experimental construction to study the behavior of expansion and contraction joints at different spacings was initiated in 1940 and 1941, and the results after 10 years of service were reported in 1956 (101). In summarizing the reports from the various states, Sutherland (101) stated:

"It was found that in pavements with expansion joints spaced at what was considered to be a desirable interval and intermediate contraction joints at sufficiently close intervals to control transverse cracking there was a tendency for the expansion joints to close progressively and the contraction joints to open progressively with time. These movements progress rapidly during the early life of the pavement and within a few years are of sufficient magnitude to destroy aggregate interlock in contraction joints of the weakened plane type. Where the expansion joints were eliminated or widely spaced there has been little or no tendency for the contraction joints to open progressively.

"On the basis of the 5-year progress reports the practice of many of the states with respect to expansion joints has changed. Today practically every state has eliminated expansion joints in nonreinforced concrete pavements except at structures and other special locations. This has resulted in pavements which offer greater resistance to pumping and faulting because of the better maintenance of aggregate interlock in the contraction joints."

In a study of the structural efficiency of transverse weakened-plane joints, Sutherland and Cashell (102) concluded that aggregate interlock in weakened-plane joints could be expected to break down under the severe hammering of heavy, high-frequency traffic and that where stress control is desirable it is necessary to use some more effective device for load transfer.

Load transfer devices, generally in the form of round, smooth steel dowels, are required at all transverse joints between long reinforced slabs. In 1959 Teller and Cashell (103) reported the results of laboratory tests to develop information on the structural action of load transfer systems under repetitive loading. Of immediate application was their finding that for round steel dowels at a 12-in. spacing in joint openings of ¾ in. or less, the dowel diameter in eighths of an inch should equal the slab depth in inches.

Reinforcement—Reinforcement has been involved in all accelerated tests under controlled traffic from the Bates Road Test (2) to the AASHO Road Test (56). This included Road Test One-MD and tests on airfield pavements at Lockbourne (59, 60) and Sharonville (62). In each of these investigations except Road Test One-MD, both reinforced and nonreinforced pavements were subjected to the prescribed traffic. In all cases except for the AASHO Road Test, the reinforced pavements generally performed better than the nonreinforced.

Under the AASHO Road Test conditions there was no significant difference between the performance of the reinforced and nonreinforced pavements. However, load transfer dowels were used at the joints between the 15-ft nonreinforced slabs as well as between the joints for the 40-ft reinforced slabs. These have been absent from the nonreinforced pavements in other previous investigations.

In the Bates Road Test and the tests at Lockbourne and Sharonville a common finding was that the reinforcing prolonged the life of the pavement after the first crack occurred. An analysis of the Lockbourne and Sharonville tests demonstrated that the steel reinforcement increased the effective thickness of the rigid pavement in proportion to the percentage of steel used in the design.

Based on a very comprehensive survey covering several thousand miles of highways in service, Hogentogler (104) reported in 1925 on the economic value of reinforcement in rigid pavements. Further information on performance of reinforced pavements is reported by Spencer, Allen and Smith (77), by Vogelgesang and Teske (105) and by Velz and Carsberg (106). A significant observation from these surveys and experiments is that the use of distributed reinforcement in the slab prevents faulting and lessens progressive deterioration at cracks between joints. It is also indicated that reliable results are obtained if the required steel area is determined on the basis of the "subgrade-drag" formula using an average coefficient of subgrade friction of 1.5 and an allowable stress in the steel equal to ⅔ to ¾ of the yield strength of the particular grade of steel.

Rigid Overlays

Bradbury (99) reported that a composite pavement consisting of a concrete base and a grout-filled brick surface in the Bates Road Test (2) was similar in its effectiveness to resist flexure due to loads to an equivalent thickness of concrete pavement in accordance with the formula $h^2 = h_b^2 + h_s^2$ where h is the equivalent thickness and h_b and h_s are the thicknesses of the concrete base and the grout-filled brick surface respectively.

Although a number of rigid overlays had been constructed over the years, it was not until the Corps of Engineers (100) initiated a program in the 1950's that data were obtained to formulate design criteria for overlay pavements. This program included theoretical studies and actual field tests. As a result of these investigations the above equation was modified to cover two conditions as follows:

Partial bond: $$h_o^{1.4} = h_d^{1.4} - Ch^{1.4}$$

No bond: $$h_o^2 = h_d^2 - Ch^2$$

where

h = thickness of base slab,
h_o = overlay thickness,
h_d = equivalent single-slab thickness, and
C = coefficient relating to the condition of the base slab.

Continuously Reinforced Pavement

Experimental continuously reinforced concrete pavements were constructed in Indiana (1938), New Jersey and Illinois (1947), California (1949) and Texas (1951). Since these early experiments a considerable mileage of additional experimental pavements as well as standard construction has been built (97). The experimental data and service behavior surveys indicate that the optimum amount of longitudinal steel is that which results in a crack spacing of from 3 to 10 ft. A closer spacing is not necessary and a spacing greater than 10 ft is not desirable. Also, it has been found that crack intervals within this range may be obtained with a steel area of from 0.5 to 0.7 percent of the cross-sectional area of the slab.

Performance studies reveal that (a) the numerous fine transverse cracks are usually not visible to the driver or passenger and that crack widths remain insignificant under all conditions and sealing is not necessary; (b) although the edges of the cracks were under traffic, this is only a surface condition which does not affect the load-carrying capacity or the riding qualities; (c) there is no spalling, faulting or pumping at the tightly closed cracks; and (d) the cracks are held tightly closed, which prevents the passage of water and the infiltration of solid material.

An important finding from the various studies is that longitudinal movements are limited to the end sections (200 to 500 ft) and that the long central portion is, for all practical purposes, fully restrained. This is true regardless of the uninterrupted length of the pavement. Various states and organizations (107, 108) have developed design practices which cover design details such as the calculation of required steel areas, size and spacing of members, lap requirements, construction joints and terminal joints.

Prestressed Pavements

The state of the art of prestressed concrete pavements has been presented quite comprehensively in the report (55) of the Subcommittee on Prestressed Concrete Pavements of the HRB Committee on Rigid Pavement Design. Practically all prestressed concrete pavements up to the present time are to be found in Western Europe and most of these were constructed in the late 1950's. Except for reports that these pavements are performing satisfactorily, no detailed performance data concerning them were available when the Subcommittee report was prepared.

Design theories and construction practices have been developed to the point where prestressed pavements can compete economically against conventional concrete pavements in certain countries in Europe—especially on airports where the reduction in pavement thickness can be more than 50 percent. This reduction could probably apply to highway pavements as well, but because of necessary construction procedures a minimum thickness of about 5 in. seems to be required.

The construction of the few relatively short sections of prestressed concrete pavement thus far in the United States has been rather costly, probably because this is a new concept and there is a lack of construction experience. In addition to the customary construction equipment and processes, a prestressed pavement requires (a) special jointing since transverse joints may be 300 to 800 ft apart, (b) sleeper slabs under the joints, (c) friction-reducing layers between the pavement and the subgrade or subbase, (d) steel tendons and conduits distributed throughout the area of the pavement, (e) filling the conduits with grout after the tensioning is completed, and (f) jacks for stressing the tendons.

An empirical equation has been developed (109) for predicting the top surface cracking load of a prestressed pavement. This equation was based on results of laboratory load tests on 45 small-scale prestressed slabs constructed of gypsum cement. Both pre-tensioned and post-tensioned slabs were loaded statically with loads simulating single-wheel and multiple-wheel gear configurations. During testing, the slabs were supported by a rubber subgrade that had a reaction modulus of 35 pci. The results indicate that, for a single static application at an interior location, the top surface cracking load of a prestressed pavement is dependent on the radio of the radius of the loading area to the radius of relative stiffness.

Because a prestressed pavement will be subjected to many repetitions of moving loads during its lifetime, pavement fatigue must be considered, especially at the working bottom surface cracks. The life of a prestressed pavement subjected to repeated moving loads at interior locations was studied (110) during tests on seven precast, pre-tensioned laboratory model slabs. These tests were designed to determine the relationship between prestressed pavement performance under repetitive moving loads and performance under single static loads as previously established by the empirical equation. Initial visual distress from traffic loads consisted of top surface circular cracking patterns in the traffic area. There was little or no evidence of pavement distress prior to the development of surface cracking. After the circular crack patterns occurred, a few additional repetitions of load generally were sufficient to completely disintegrate the concrete for the full slab depth in the distressed area. Results of these model tests were evaluated to determine design factors for repetitive loading. A graph of design factors vs interior load coverages producing pavement failure was presented for use in predicting prestressed pavement traffic life for different combinations of design variables.

HRB Special Report 78 reviews the progress that has been made during the years since the initial construction of a prestressed airfield pavement (55). It is evident that further improvements are needed, from the standpoint of both design and construction, before all the advantages of this type of pavement can be realized. However, experience in Europe demonstrates that existing difficulties can be overcome and that current design and construction practices can be used with the assurance of satisfactory results.

There is a considerable amount of accomplished research and service experience on concrete pavements that is not included in the reference section of this report. Two possible sources for this additional material would be the U.S. Bureau of Public Roads Office of Research and Development and the Highway Research Information Service (HRIS) of the Highway Research Board.

SOME CURRENT DESIGN PROCEDURES

Because of the wide range of environments and materials encountered in pavement construction it is not surprising that many methods are available for the thickness design of concrete pavements. Most of the methods combine a theoretical determination of the load stresses with an experimental or theoretical approach to the fatigue of concrete. However, in some cases the theoretical analysis has been supplemented or modified by experience gained from field observations of the performance of in-service pavements. To illustrate some of the considerations involved in the thickness design of concrete pavements, a brief description is given of some current design procedures. References to a particular design method should not be considered an endorsement of one method over others, nor should it be construed that a method not included was considered inadequate.

Plain Concrete Pavements

American Association of State Highway Officials—The empirical equation derived from the AASHO Road Test (111) provides a basis for design that incorporates axle load, number of load applications and slab thickness. The equation was derived for fixed values of modulus of elasticity, modulus of rupture, modulus of subgrade reaction, joint design, environmental conditions and test period. The equation expresses the ratio of the loss in serviceability at any time to the total potential loss. The basic equation was extended for general use by incorporating the semi-empirical stress equation developed by Spangler. The Spangler equation is used to linearize the measured strains, and the two combined equations yield a general design equation. The Spangler equation applies to a load at a corner, and for development of the Road Test equation the distance from the corner to the center of the load was taken as 10 in. The dense liquid subgrade theory is assumed, and the strength of the subgrade is expressed by the modulus of subgrade reaction, k. The design nomographs assume constant values of E = 4,200,000 psi and M = 0.20. However, these values can be changed in the design equation. The allowable working stress is set at 75 percent of the modulus of rupture. To simplify the equation a standard single-axle load of 18 kips is used. For other axle loads, conversion factors are provided to convert any given load into equivalent 18-kip single-axle loads. The fatigue of the pavement is considered indirectly in the concept of serviceability by including an estimate of the number of load applications for the design road life. No special provision is made for impact or a safety factor. At present, the design procedure does not include a regional factor. The nomographs are based on a design life of 20 years. For a road life other than 20 years, the daily equivalent load applications are multiplied by a factor equal to the design life in years divided by 20 years.

American Concrete Institute—Recommendations are presented for the design of concrete pavements and bases (122) based on practice proved successful in the United States. Comprehensive directions are given for designing rigid airport and highway pavements or bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. Included are recommendations for soil foundations, selection of slab dimensions, joints and details for nonreinforced pavements.

Federal Government Agencies—The following agencies have design procedures for plain concrete pavements (details are contained in the references noted): Department of the Navy (73); Federal Aviation Agency (71); and Corps of Engineers (100, 120, 121—used by Departments of the Army and Air Force).

Texas State Highway Department—The Texas design method for rigid pavements (112) is basically an extension of the design developments presented in the AASHO Interim Pavement Design Guide. The variables are magnitude of load, repetition of load, tire pressure, axle type, strength of support media, concrete strength, concrete modulus of elasticity, Poisson's ratio, the continuity condition, friction of the support media and the regional factors.

The pavement structure is designed to encompass external loads introduced to the pavement from wheel loads. The following equation for pavement thickness is similar to that presented in the AASHO Guide except that it was expanded to include concrete modulus of elasticity, total traffic and pavement continuity (jointed or continuous):

$$\log \Sigma L = -8.682 - 3.513 \log \left[\frac{J}{S_x D^2}\left(1 - \frac{2.16a}{Z^{1/4} D^{3/4}}\right)\right] + 0.9155 \frac{G}{B} \qquad (11)$$

where

ΣL = number of accumulated equivalent 18-kip single-axle loads;
J = a coefficient dependent upon load transfer characteristics or slab continuity;
S_x = modulus of rupture of concrete at 28 days (psi);
D = nominal thickness of concrete pavement (inches);
Z = E/k;
E = modulus of elasticity for concrete (psi);
k = modulus of subgrade reaction (psi/inch);
a = radius of equivalent loaded area = 7.15 for Road Test 18-kip axles;
$G = \frac{P_o - P_t}{3} = \frac{4.5 - P_t}{3}$;
P_o = 4.5 = serviceability at time zero;

P_t = serviceability at end of time, t; and

$$B = 1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}$$

Only one term in the equation has not been evaluated adequately, the continuity or J term. The selection of a value J for design purposes must be postulated on the basis of limited data. The J value for the jointed pavements on the AASHO Road Test is automatically fixed at the value of 3.2 which was used in all correlation work.

Portland Cement Association—The design procedure (113) is semi-empirical, utilizing the mathematical analysis developed by Westergaard (98). The solution of the mathematical analysis has been presented in convenient influence charts (114) for determination of moments for various positions and configurations of loads. The axle is assumed parallel to a joint with no load transfer and with the tire contact areas as close as possible to the joint. Design nomographs have been developed from the influence charts for single- and tandem-axle loads. The nomographs relate stresses, modulus of subgrade reaction k, axle load and slab thickness.

In developing the nomographs, tire imprint areas were varied with the amount of load. Poisson's ratio was given a fixed value of 0.15, and the modulus of elasticity was assumed equal to 4 million psi. These values can be varied in the basic equation. Life expectancy of the pavement is determined by considering the applied stress as a function of the flexural strength of the concrete and the anticipated number of load applications. Loads causing a stress less than one-half the modulus of rupture are not considered in the design. The fatigue resistance consumed by each load level is the ratio, expressed as a percentage, of the expected load repetitions to the allowable load repetitions determined from a fatigue diagram. The sum of the fatigue resistances should not exceed about 125 percent. A fatigue resistance of greater than 100 percent is permissible because a conservative value of flexural strength is used that does not consider the gain in concrete strength with age. Load safety factors are recommended for all highways except residential streets and other light-traffic roads.

Jointed Reinforced Concrete Pavements

Wire Reinforcement Institute—Thickness design for reinforced concrete pavement, in accordance with the Wire Reinforcement Institute design procedures, is based on the theoretical analyses of Westergaard modified by empirical relationships obtained from observations of performance under service conditions (99). In this respect thicknesses are the same as for plain pavements, but the basic design includes distributed steel throughout the slab and mechanical load transfer at the joints. This design provides load transfer at all joints and at any cracks that may form between the joints.

The amount of steel, longitudinal and transverse, is determined by the "subgrade drag" formula discussed earlier, which takes into account (a) the length of the slab between free joints, (b) the coefficient of friction between the slab and subgrade or subbase, (c) the weight of the slab, and (d) the allowable stress in the steel. The steel is intended to hold cracks tightly closed and thus maintain the shearing resistance of the slab at a crack. This permits the use of longer slabs (40 to 100 ft) than can be permitted in plain pavements.

American Concrete Institute—Recommendations are given for the design of jointed reinforced concrete pavements and bases (122) based on practice proved successful in the United States. (This is the same reference as shown for plain concrete pavements.)

Texas State Highway Department—Steel reinforcement is used in concrete pavement where the joint spacing exceeds 15 ft. The reinforcement is placed in the slab to hold any cracks that form in the pavement tightly closed. The pavement may thus perform as an integral structural unit. The Texas method covers the design of two of the three basic types of reinforced concrete pavement, i.e., jointed reinforced and continuous reinforced (prestressed concrete pavement is not covered).

The equation for thickness design is the same as used for plain pavements (see Eq. 11). A value of 3.2 is used for the J term. The reinforcement for the jointed concrete pavement is determined by the application of the conventional subgrade drag theory. In essence, the equation is based on the principle of balancing the slab resistance to movement against tensile strength of the steel. The formula is

$$P_S = \frac{L\,F}{2\,f_S} \times 100 \qquad (12)$$

where

P_S = required steel percentage, percent;
L = length of slab between joints, feet;
F = friction factor of subbase; and
f_S = allowable working stress in steel, psi.

Portland Cement Association—The PCA design procedure for reinforced concrete pavement does not consider distributed steel to increase the flexural strength of an unbroken slab when used in quantities considered to be within the range of practical economics. The principal functioning of the steel is to hold together the fractured faces of slabs after cracks have formed. Hence, the thickness design procedure of the concrete is the same as for plain concrete pavements. The amount of distributed steel required is based on (a) distance between joints, (b) coefficient of friction between slab and subgrade, (c) the weight of the slab, and (d) the allowable working stress in the steel.

American Association of State Highway Officials—The AASHO procedure does not specify a method for design of thickness of the concrete. The amount of steel is determined by the subgrade drag theory.

Departments of the Army and Air Force—The advantages in using steel reinforcement include a reduction in the required slab thickness. The design procedure of the Corps of Engineers has been developed empirically from a limited number of prototype test pavements subjected to accelerated traffic testing and performance data. Although it is anticipated that cracking will occur in pavements under the design traffic loadings, the reinforcing will hold the cracks tightly closed, which prevents spalling and faulting and provides a serviceable pavement during the anticipated design life. The designer determines the percent of steel required, the thickness of the reinforced rigid pavement and the maximum allowable length of the slabs. The Corps of Engineers uses the following relationship to determine the increase in effective slab thickness h in percent as a function of the amount of steel S each way as a percent of the area:

$$h = 16.0 \times \log_e S + 50$$

This relationship is shown graphically as Figure 8 in the paper by Mellinger et al (62) and has been found applicable also to vehicular traffic.

The maximum allowable slab width or length on Corps of Engineers projects is determined by the equation

$$L = [0.00047\, h_r\, (f_S S)^2]^{1/3}$$

where h_r is the thickness of the reinforced pavement in inches, f_S the yield strength of the reinforcing steel in psi, and S the percent of reinforcing steel.

It should be noted that, while equal percentages of steel are used in both transverse and longitudinal directions of airfield slabs, vehicular pavements were found to perform satisfactorily for their design life with the percentage of steel used in the transverse direction equal to one-half that used in the longitudinal direction.

The design criteria for reinforced pavements were found by the Corps of Engineers to be subject to the following limitations:

1. No reduction in the required thickness of nonreinforced rigid pavements should be allowed for $S < 0.06$.
2. No further reduction in the required thickness of nonreinforced rigid jointed pavements should be allowed over that computed for $S = 0.50$ regardless of the amount of steel used.
3. The length L should not exceed 75 ft for vehicular and 100 ft for airfield pavements regardless of the values of S, h_r or f_s for jointed pavements.
4. The minimum value of h_r should be 6 in.

Continuously Reinforced Concrete Pavements

American Association of State Highway Officials—The Guide (111) provides an equation for the determination of a theoretical minimum percentage of steel based on (a) the coefficient of friction between slab and subgrade, (b) the allowable working stress in the steel, (c) the tensile strength of the concrete, and (d) the elastic modulus ratio of the steel to that of the concrete.

Under severe temperature variations a formula is suggested which considers (a) the thermal coefficient of concrete and steel, (b) the temperature range, (c) the coefficient of friction between slab and subgrade, (d) the allowable working stress in the steel, (e) the tensile strength of the concrete, and (f) the modulus of elasticity of the steel.

Wire Reinforcement Institute—The selection of pavement thickness is based on observations of the performance of continuously reinforced concrete pavements under service conditions. It takes into account route classification, locality description, controlling vehicle type, and controlling single-axle load. On the basis of these factors, recommendations for pavement thickness vary between 6 and 9 in. In this connection it is suggested that the design methods under development by AASHO be used when they become available. The interim AASHO Guide (111) is an example of a more rational design.

The amount of longitudinal steel, based on a yield strength of 70,000 psi for deformed steel welded wire fabric, is recommended as 0.5 to 0.7 percent of the cross-sectional area of the slab. A proportionate adjustment in percentage is recommended for steels having other yield strengths. The area of transverse reinforcement is determined in the same manner as for jointed concrete pavements.

Recommendations are given for spacing of longitudinal and transverse members, and it is recommended that the steel not be placed below the mid-depth of the slab and that the concrete cover be not less than 2 in.

Concrete Reinforcing Steel Institute—The CRSI design procedure (108) is semi-empirical and utilizes the mathematical analysis developed by Westergaard (3). The equations have been given in influence charts (114) that may be used in the design.

The design procedure assumes that the load is applied at an interior position. The subgrade is assumed to be a dense liquid and its strength is expressed by the modulus of subgrade reaction, k. Design charts are shown based on the assumption that the tire imprint area is a function of the wheel load, and the loaded area is assumed equal to the area of contact of the dual-wheel tires plus the area between them. The equivalent radius is the radius of the loaded area as defined. Poisson's ratio is assumed as 0.15 and the modulus of elasticity is taken as 4 million psi. An allowable working stress of 50 percent of the modulus of rupture is suggested for use. The design wheel load is taken as the design static wheel load increased by a 20 percent load factor allowance. Fatigue is implicitly considered by permitting a working stress of 50 percent. The 20 percent load factor allowance provides for an impact or safety factor. No regional factor is considered.

The amount of steel is based on accepted minimum criteria of (111) 0.7 percent longitudinal steel for a 7-in. slab or (112) 0.6 percent for an 8-in. slab. Deformed reinforcing bars ¾ in. (No. 6) or less in diameter with a minimum yield point of 60,000 psi are recommended.

The reinforcement may be located at a depth from 2½ in. below the surface to mid-depth of the slab. If longitudinal bars are staggered, they should be lapped a minimum of 25 bar diameters; if not staggered, 30 bar diameters.

Texas State Highway Department—Thickness design is based on the same equation used for plain pavements, but a continuity or J value of 2.2 has been established based on comparisons of previous design procedures and performance studies. The design method for continuous reinforced concrete is based on the concept of balancing the internal concrete stresses developed by temperature and shrinkage against the strength of steel. The formula for determining the minimum percent longitudinal steel is

$$P_s = (1.3 - 0.2F) \frac{S'_c}{f_s - NS'_c} \times 100 \tag{13}$$

where

P_s = required steel percentage;

F = friction factor of subbase;

S'_c = tensile strength of concrete, psi;

f_s = allowable working stress in steel, psi (0.75 of yield strength recommended, the equivalent of safety factor of 1.33),

E_c = modulus of elasticity of concrete, psi,

E_s = modulus of elasticity of steel, psi; and

N = E_s/E_c.

PROPOSED AREAS FOR FUTURE RESEARCH

After considering the research now in progress and the overall problem of pavement design, the Rigid Pavement Committee proposes several areas of study for future research. The Committee recommends that any agency conducting research on rigid pavements give serious consideration to encompassing or partially encompassing any of the research problem areas listed.

The various areas of research are outlined as research problem areas with subdivisions describing the problem and defining the objectives of the proposed research.

Research Problem Area No. 1

Problem—Several very significant changes in the design and construction of rigid pavements have taken place in recent years. Evaluation of the effects of these changes has generally been on a state basis or limited to experimental projects. Consequently, a critical need exists for a nationwide appraisal of the performance of postwar rigid pavements, with emphasis on diagnosis of pavement defects.

Objectives

1. Identify and describe the significant changes that have taken place in pavement design in the past 15 years, such as subbase requirements, pavement type, jointing practices, and sealants.
2. Characterize possible defects that may occur in these pavements.
3. Develop guidelines that will provide for (a) a nationwide study of the nature, extent, seriousness and cause of pavement defects, and (b) correlation of these defects

with construction materials, structural components, climate, traffic, soil conditions and other factors.

Research Problem Area No. 2

Problem—Current analysis procedures of pavements subjected to dynamic loads are inadequate. Current methods of basing so-called repetitive load designs on fatigue life of concrete are not satisfactory. Information made available by repetitive load tests such as the AASHO Road Test, while very helpful, has not provided adequate solution to the problem.

Objectives—Establish analysis techniques that will provide a coordinated evaluation of repetitive load variables in pavement design. Utilize these analysis techniques to evaluate existing information, such as that available from full-scale traffic tests (AASHO Road Test) and repetitive load tests on model pavements. Finally, based on these analyses, make recommendations for additional physical testing or other work required to provide the necessary information for correct dynamic design of pavements.

Research Problem Area No. 3

Problem—It is postulated by some that the effect of environment on rigid pavement performance far outweighs the effect of all other variables involved, including the effect of repetitive loads. This is particularly true in the northern part of the United States. Therefore, there exists a great need to evaluate the effects of environment, including climatic and regional variables (moisture and temperature), on pavements on a nationwide basis.

Objectives—Establish the magnitude of various environmental effects on rigid pavement performance. These effects are known to include temperature and moisture as well as the general effects of location which have been termed regional effects in NCHRP Report 2A (4).

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State of the Art of Skid Resistance Research

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•ALL AGENCIES responsible for the construction and maintenance of roads and streets are faced with the problem of providing pavements on which adequate frictional resistance can be developed by vehicle tires. This frictional resistance is necessary to start, turn, accelerate, and stop a vehicle. The requirements are similar on airfields for the take-off and landing of airplanes.

Although the frictional resistance on modern roads is being gradually improved by appropriate construction and maintenance measures, the problems of the highway engineer are still unsolved because the required standards of frictional resistance are going up at an even faster rate due to the steadily growing volume of traffic and the increasing speed and performance of modern vehicles.

The final goal of any research effort in this field is to build roads that offer adequate pavement-tire friction under modern traffic conditions. To achieve this, it is necessary (a) to understand the fundamental concepts of road friction, (b) to measure this property effectively, and (c) to evaluate the results in terms of the individual factors involved. In this paper past research results, present research trends, and future research needs are discussed under these three headings.

PAST RESULTS

Highway engineers have been studying the problem of vehicle skidding and the resistance to skidding offered by pavement surfaces for several decades. In fact, early technical papers on the subject, which perhaps somewhat surprisingly touch on many of our present-day ideas, are about 30 to 40 years old. Other engineers (notably mechanical and aeronautical engineers), other scientists (notably physicists and geologists), and various other groups (automobile manufacturers, tire manufacturers, etc.) have also joined in the work. Understandably, only the major findings can be covered in this paper, and only in rather general terms.

A good percentage of the research results has been published under the auspices of the Highway Research Board, the American Society for Testing and Materials, the Permanent International Association of Road Congresses, and the British Road Research Laboratory. In order to correlate individual research efforts, an international conference (the First International Skid Prevention Conference) was held at the University of Virginia in 1958. This conference was successful in establishing contacts and working relationships among the leading investigators from several countries. Much of the present knowledge on skidding and skid resistance is consolidated in the two-volume Proceedings of the conference (12).

In addition to the United States, the major centers of skid resistance research have been Britain, Germany, Sweden, and France. Research reports from these countries and from Australia, Belgium, Canada, Czechoslovakia, Denmark, Holland, Israel, Italy, Japan, Poland, Russia, Spain, and Switzerland were covered in a literature review. This first section of the present paper is essentially a digest of that literature review. Past results of general interest are discussed here under three headings: nature of skid resistance, measurement of skid resistance, and skid resistance parameters.

Nature of Skid Resistance

The forces that are required in driving, steering, and braking an automobile are provided by the frictional resistance developed between the tires and the pavement surface. For any given tire and pavement combination there is a maximum friction potential; if it proves to be inadequate for the intended maneuver, the wheels begin to slide, the driver's control over the vehicle is drastically reduced, and the vehicle is said to be "skidding." The frictional resistance offered by the pavement surface to the sliding tires is called "skid resistance."

The most common example of vehicle skidding occurs during a panic stop, when all the wheels are locked and cease to rotate but the vehicle continues to slide. The implications of adequate skid resistance with respect to traffic safety are fairly obvious.

Skid resistance is usually considered to be a frictional force consisting of two components—the adhesion term and the deformation term. The adhesion term is the result of molecular forces, and its magnitude is determined by the nature of the two materials in contact, in this case the tire and the road surface, and by the normal force between them. Adhesion is usually the dominating factor on dry pavements; its effect diminishes with lubrication and becomes negligible on wet pavements. On wet pavements the deformation term is far more important. It is the result of the deformation of the tire rubber by the surface asperities and the ensuing energy dissipation in the rubber.

Under any given set of conditions the effective skid resistance (F) is the sum of the adhesion (F_a) and deformation (F_d) components; dividing it by the vertical load (L), it can be expressed in terms of an effective coefficient of friction (f): $f = F/L = (F_a + F_d)/L$. Since the adhesion term is greatly reduced (while the deformation term is essentially unaffected) by lubrication, the skid resistance of a pavement is always lower in the wet state than in the dry state. Typically, the difference may be of the order of 50 percent, depending on surface texture (higher on smooth pavements than on coarse ones) and testing speed (higher at high speeds than at low speeds). The great majority of wet-road skid resistance measurements are in the 0.30 to 0.70 range.

Measurement of Skid Resistance

In order to insure critical test conditions, skid resistance measurements are normally taken on wet pavements. Unfortunately, this has turned out to be practically the only standard feature of the procedures for measuring skid resistance. A number of diverse techniques and devices have been developed, and they each appear to have certain advantages. For the purposes of this discussion the various skid resistance measuring devices have been grouped into four classes—braking force trailers, other braking force methods, the sideway force method, and portable and laboratory instruments.

It should be noted that the indirect evaluation of skid resistance, by observation of the relative frequency of skidding accidents at certain sites, has also been attempted. Such data have been used to assess the significance of the problem of skidding, to validate skid resistance measuring techniques, to study the effect of individual skid resistance parameters, to locate accident black-spots, and to check the efficiency of corrective surface treatments.

Braking Force Trailers—This apparatus consists of a trailer with one or more test wheels and a towing vehicle which usually carries the recording instruments and a water tank. As the brake is applied on the test wheel, constant towing speed is maintained and the retardation forces acting in the plane of the test wheel are measured. Knowing the vertical load, the effective coefficient of friction can then be calculated.

If the brake is gradually applied on the test wheel, the percentage slip will gradually increase. (Percentage slip is defined as $s = (N_f - N_b) \cdot 100/N_f$, where N_f is the revolution rate of a freely rotating wheel and N_b the revolution rate of the braked wheel.) Generally, the measured frictional resistance will also increase up to a peak value obtainable at a critical slip percentage, and then it will decrease to the value of locked-wheel or sliding friction. The peak value of frictional resistance is usually called the "impending" or "incipient" friction; it is obtained at 5 to 20 percent slip, and it is likely to be about 50 to 100 percent higher than the sliding friction value.

The force-measuring instrumentation varies with trailer type, both in basic concept and in degree of sophistication. The parallelogram-type and torque-type systems are considered to be the best. Normally the coefficient of sliding friction is measured during automatically controlled brief periods of braking, and the results may be graphically recorded. Some trailers have been constructed to measure impending friction at a predetermined slip value.

Trailer tests provide a simple and direct approach to the problem of measuring skid resistance, they can be carried out at normal traffic speeds with minimum interference to traffic, and the procedure is fast and accurate (accuracy about ± 1 to 2 percent). While the initial cost of the equipment is relatively high, the unit testing cost (cost per test site) is usually very reasonable.

This testing technique has now become the most popular one in the United States, and over a dozen different skid-trailers are being used for routine testing. Advanced skid-trailers have also been developed in Sweden, Germany, Britain, France, Italy, and other European countries.

Other Braking Force Methods—Another technique based on the braking force concept is the stopping distance method. The test vehicle is accelerated to a predetermined initial speed, its wheels are then locked and the distance required for the vehicle to stop is measured. The effective sliding friction, corresponding to the speed range from the initial speed to zero speed, can then be calculated using the simple formula $f = V^2/30\,S$, where V is the initial velocity in mph and S is the stopping distance in feet.

Normally, slightly modified standard passenger cars are used for stopping distance tests. The stopping distance is measured either by means of a stopmeter with a fifth wheel (both the initial speed and the stopping distance being recorded on dials inside the car), or with a tape from a chalk-mark made by a brake-activated detonator. The braking system may also be modified in order to standardize braking conditions.

This is a relatively inexpensive testing method which closely simulates the critical conditions of emergency braking, and may be made realistic to the extent of reflecting changes in vehicle and tire design. On the other hand, it is normally limited to speeds below 30 to 40 mph, because there is an element of accident hazard involved and elaborate traffic control measures are required. The reproducibility of the measurements varies from ± 2 to ± 15 percent.

The stopping distance method has never been popular in Europe; it was favored by a majority of United States investigators up to a few years ago but is now gradually losing ground to the trailer method.

A decelerometer is used in a similar testing technique. As the brakes are applied, the deceleration of the vehicle is measured instead of the stopping distance. The coefficient of sliding friction is numerically equivalent to the deceleration expressed in g's. Actually, impending friction may also be measured if the brakes are gradually applied. The decelerometer is positioned either on the floor or on the dashboard of the test car.

The decelerometer method was developed in Britain, and has also been extensively applied in the United States. It is a satisfactorily accurate (typical reproducibility ± 2 to 3 percent), inexpensive, fast, and realistic testing technique, but it does require careful traffic control, and the testing procedure is rather difficult to standardize.

Sideway Force Method—The sideway force method is based on the contention that the critical maneuver with regard to skidding is cornering. The test wheel is set at a predetermined angle to the direction of travel, and the sideway force acting normal to the plane of the wheel is measured. The frictional resistance is then expressed in terms of the sideway force coefficient which is the ratio of the sideway force to the vertical load.

The maximum sideway force is obtained at a critical angle of inclination of the test wheel. This critical angle is a function of pavement surface parameters, and is usually slightly below 20 degrees. The instrumentation of the test vehicle usually involves electrical strain gages for measuring the axial force and an automatic system to obtain a continuous graphical representation of the test results.

The sideway force method is accurate (reproducibility ± 1 to 2 percent), yields continuous measurements along the road, and there are no accident hazards involved. Depending on the degree of sophistication aimed at, the initial equipment costs may be relatively high. The technique has been developed and extensively used in Britain and France, and is also being applied elsewhere in Europe. It has been rarely used in the United States.

Portable and Laboratory Instruments—A number of devices have been developed either to measure the frictional properties of pavement specimens in the laboratory or to serve as a link between field and laboratory investigations by being adaptable to testing conditions in both environments. Among them are the pendulum-type devices, the single-wheel-type devices, and the skid-cart testers.

These are specialized instruments, used normally for special purposes, such as the laboratory testing of potential paving mixes, or taking spot-checks at a busy intersection. It is always open to question how well such devices simulate the actual interaction of vehicle tires with the road surface; at best they can only indicate the skid resistance for a very limited range of operating conditions.

Skid Resistance Parameters

The main goal of all the research effort utilizing this large variety of equipment has been to discover and evaluate the parameters that significantly contribute to the skid resistance of pavement surfaces. While quantitative expressions are not yet available, a reasonably good understanding of the problem has been achieved in qualitative terms.

The skid resistance offered by a pavement surface to a vehicle tire is determined by three groups of parameters: (a) those associated with the pavement surface, (b) those associated with the vehicle, and (c) those associated with operating conditions.

Within each group, there are primary (important) and secondary (not so important) parameters. The primary parameters are: (a) aggregate characteristics and surface texture in the first group, (b) rubber properties and tread pattern in the second group, and (c) temperature and vehicle speed in the third group.

In very general terms, skid resistance will be high if effective instantaneous drainage can take place in the contact region (high adhesion term) and high hysteresis losses are provoked in the tire rubber (high deformation term). Each of the primary parameters has a decisive influence on either one or both of these basic factors of skid resistance.

Aggregate Characteristics—To be able to puncture thin "squeeze-film" between the tire and the pavement, and to cause appreciable grooving of the tire rubber, the aggregates in a paving mix should be angular, hard, and polish-resistant under wear. Opinions differ on particle size and gradation characteristics.

Numerous experiments have proved the importance of particle shape. Using the same aggregate type, skid resistance is normally higher on bituminous mixes with angular aggregates than on mixes with rounded aggregates.

It is not sufficient for the aggregates to be sharp and angular initially; they should also retain these characteristics in service. Experience shows that considerable reductions in skid resistance may be caused by traffic wear—as much as 50 percent of the initial skid resistance is lost on some pavements during the first two years of service. Much of this is caused by the polishing of the individual aggregate particles in the mix. The rate of deterioration is considerably slower in the later stages of service, and polishing practically levels out once the "ultimate state of polish" is reached. Heavy and fast vehicles cause more rapid reduction of skid resistance than light and slow ones; under heavy traffic the ultimate state of polish may be reached as early as two years after construction. (It may be noted that a number of accelerated wear machines and procedures have been developed to simulate the effect of traffic wear on laboratory pavement specimens.) Polishing always requires the presence of some abrasive material, such as the clay, silt, and sand-sized dirt and dust usually covering road surfaces.

In order to resist polishing, the individual aggregate particles should be hard. Calcite, with a Mohs hardness of 3, is susceptible to polishing by just about any ingredient of the road scum; quartz has a Mohs hardness of 7, and the only mineral abundantly present in the road scum that would be hard enough to cause it to polish is quartz itself.

It is therefore not surprising that limestones are often the main cause of slipperiness on both flexible and rigid pavements. (Certain limestones, it must be added, particularly the highly dolomitic formations and those having a relatively high acid-insoluble sand-size content, have given entirely satisfactory performance.) Sandstones and slags are usually regarded as good anti-skid aggregates. Sandstones give a good example of differential wear, which is a desirable kind of wear. Due to the difference in hardness between the hard crystals and the friable matrix, traffic wear actually contributes to a rough, uneven surface texture on sandstone aggregates.

Surface Texture—A sharp, gritty, "deep" surface texture, with adequate channels and escape paths among the asperities, facilitates the escape of the lubricating water from the contact patch, and is therefore an important factor in providing good skid resistance.

The large-scale texture (roughness) of a pavement surface may be important in breaking up the contact patch into smaller areas that drain faster. This effect is definitely significant for smooth tires; with patterned tires, however, the roughness of the surface may become a secondary factor since the tire pattern itself can adequately subdivide the contact area. What is always important is the small-scale texture of the surface, i.e., the sharpness of the very small-scale projections and ridges (harshness). Experience has shown that road surfaces with "sandpaper texture" usually exhibit high skid resistance.

The possible role of differential wear in retaining a harsh or rough surface texture has been mentioned before. Another similarly beneficial type of wear is that observed with Kentucky rock-asphalt and silica-sand surface treatments. The individual aggregate particles are dislodged from the surface before they get excessively polished, and other, unpolished particles become exposed.

Pavement surface textures have been recorded by photographic and ink-printing techniques, and the sand-patch method has been used to determine texture depth.

Other Pavement Factors—Pavement type is not regarded as a critical factor; good skid resistance may be achieved with any modern pavement structure. Skid resistance appears to vary over a wider range on bituminous pavements than on portland cement concrete pavements, probably because of the more effective polishing of the aggregates. An advantage of bituminous pavement construction is that polishing aggregates may safely be used in lower layers, and wear-resistant aggregates need only be used in a thin surface course. With portland cement concrete pavements, surface finish is of considerable importance. Broom or burlap-drag finishing usually provides good initial skid resistance, although it may be necessary to provide deeper texture on high-speed roads.

Mix proportions and the properties of the binder may also be of some significance. In bituminous pavements excessive asphalt content and improper aggregate gradation may lead to bleeding; bleeding bituminous surfaces are always slippery, with friction coefficients as low as 0.10 to 0.20. On portland cement concrete pavements, surface slipperiness may be due to excess sand or water in the mix, or to a finishing carried out with excess cement paste.

Surface contamination (loose dust and grit on the pavement) is of limited consequence on wet roads, but it may reduce the skid resistance of dry pavements by as much as 50 percent. The presence of the so-called "traffic film" (an accumulation of oil, worn rubber, and dust) is usually regarded as a sign of increased slipperiness. Ice or snow cover naturally reduces skid resistance; typical measurements may be 0.05 to 0.20 on ice-covered roads, and 0.15 to 0.35 on packed snow.

Due to intensive polishing action, curves, roundabouts, bus stops, steep slopes, etc., are usually associated with slippery conditions.

Rubber Properties—It has only been a relatively recent development in the history of skid resistance research to focus attention on the properties of the tire rubber, most notably on its hysteresis characteristics. It has been concluded that the deformation component of skid resistance is due to hysteresis (internal friction) losses within the tire rubber as the deformations caused by the surface asperities are being recovered. High hysteresis (low resilience) rubber tires will therefore provoke higher frictional resistance, particularly on rough surfaces where the deformation term is especially significant. Modern "dead-rubber" (high hysteresis) tires provide about 20 percent improvement in skid resistance on rough surfaces. In order to avoid the overheating of the tire, a duplex structure may be used, with the tire tread built of high-hysteresis rubber and the rest of the tire of low-hysteresis rubber.

The hardness of the tread rubber may also have some bearing on skid resistance, especially on ice and other very smooth surfaces. The harder the tread rubber the smaller the contact area, resulting in higher localized pressures, better drainage, and increased frictional resistance.

Tread Pattern—It has been pointed out by several investigators that the friction coefficients measured with smooth and patterned tires differ considerably, possibly by as much as 50 percent. A tire with a good tread design, i.e., with circumferential ribs and transverse slots, will normally give higher skid resistance than a smooth tire, especially on polished wet surfaces.

The significance of a good tread pattern is derived from its effect on drainage conditions, and is therefore restricted to wet pavements. The elements of the tread pattern effectively break up the contact patch between the tire and the pavement surface into a number of small contact spots, making it possible for the lubricating liquid to more readily escape from the contact area. The edges in the pattern also provide a wiping action, tending to remove the lubricating film. Naturally, such improvements in drainage conditions are more effective on smooth surfaces than on rough surfaces. Similarly, patterned tires are more effective in improving skid resistance at high speeds than at low speeds, since at high speeds drainage is a more critical factor.

The role of tire characteristics in providing adequate skid resistance has been recognized by the tire industry. During the past decade, improvements in tire composition and design have led to a gradual improvement in average wet-road skid resistance on passenger-car tires.

Other Vehicle Factors—Laboratory tests have shown that the coefficient of friction is independent of the shape or size of the contact area but is inversely related to the contact pressure. Since with increasing wheel load or increasing inflation pressure the contact pressure also increases, it is not surprising that several investigators have observed a trend for skid resistance to be reduced at higher wheel loads and/or inflation pressures. In normal passenger car operation, however, these effects are of little significance.

The size of the wheel may also be of some interest. Test wheels of very small diameter may give rise to reductions in measured skid resistance.

Finally, skidding and skid resistance measurements are also influenced by the method of braking. The vehicle will tend to keep its straight course if all its wheels are locked simultaneously or if only the front wheels are locked, but will tend to go into a spin if only the rear wheels are locked.

Temperature—Among the parameters associated with operating (traveling or testing) conditions, temperature and vehicle speed have the most profound bearing on skid resistance.

It has been widely observed and reported that skid resistance tends to decrease with increasing temperatures (ambient, pavement, and tire temperatures). A typical reduction in skid resistance might be 0.02 for a 10 F rise in air temperature. This phenomenon has been recently traced back to the hysteresis characteristics of the tire rubber: with increasing temperatures the hysteresis losses in the rubber are reduced, resulting in lower frictional resistance. On very smooth surfaces (such as a glass plate), where the effect of hysteresis losses is negligible, the temperature effect is also negligible. Also, since rubber hysteresis changes more rapidly at lower temperatures, skid resistance measurements are more sensitive to temperature changes at lower temperatures.

Temperature changes also have an effect on the viscosity of both the lubricating liquid and the asphalt binder. On dry pavements, tread-surface temperatures as high as 1000 F may develop during skidding, leading to rubber melting. The melted rubber effectively lubricates the road surface, reducing its frictional resistance.

It is also a matter of general experience that the skid resistance of a pavement tends to be higher in the winter than in the summer. Typical differences may be of the order of 0.10. These seasonal variations in skid resistance are believed to be partially due to temperature changes and partially to seasonal changes in the fine-scale texture (micro-roughness) of the road surface.

Skid resistance is also related to a number of other climatic factors. As an example a heavy rainfall following a dry spell may significantly improve road friction.

Vehicle Speed—The frictional resistance of wet pavements is profoundly influenced by the speed of the vehicle; with increasing speed, skid resistance is reduced, often by a considerable amount. This is especially unpleasant in view of the fact that the skid resistance requirements for a vehicle to stop within a certain distance increase as the square of the vehicle speed.

The primary factor in this context is probably surface drainage again. At higher speeds the lubricating liquid simply has less time to escape from the contact region. (The contact time is about 0.06 sec for an automobile at 60 mph.) It is thus not surprising that the speed effect is much more significant on smooth pavements (where drainage is more critical) than on rough ones, more significant with smooth tires than with patterned ones, and negligible on dry roads.

Standard skid resistance tests are usually run at 40 mph or at 60 kmph.

Under certain conditions on water- or slush-covered pavements, when the speed of the vehicle exceeds a certain critical value, a water layer will gradually build up under the tire, detaching it completely from the pavement surface. The tire will then merely skim over this water layer, practically unable to develop any braking or maneuvering forces. This is the phenomenon of hydroplaning, now well documented and demonstrated both by laboratory and field (aircraft and automobile) experiments.

Under hydroplaning conditions the weight of the vehicle is believed to be balanced by an upward hydrodynamic thrust. Formulas have been derived to calculate the approximate critical speed (hydroplaning speed) at which the hydrodynamic lift becomes equal to the weight of the vehicle.

According to a simplified but experimentally substantiated formula, the hydroplaning speed (V_h, mph) can be expressed as $V_h = K\sqrt{p}$, where p is the inflation pressure in psi and K is an empirical constant with an approximate value of 10. The implication of this formula is that a 25-psi automobile tire may be subject to hydroplaning at speeds around 50 mph, while an 80-psi truck or bus tire is safe even at speeds slightly exceeding 80 mph.

A number of parameters, in addition to vehicle speed, have a bearing on the phenomenon of hydroplaning. The danger of hydroplaning may be diminished or eliminated by reducing the thickness of the water layer below a certain critical value (0.10-0.40 in., depending on surface texture) by using tires with good tread patterns, by increasing inflation pressure (if practical), and by constructing rough, open-textured pavements.

Other Operating Conditions—A number of other traveling and testing conditions may also influence skid resistance measurements, such as the amount of water on the surface, operator habits, and instrumentation features.

The first factor mentioned deserves further amplification. It is normally found that the first traces of the water on the pavement (e.g., merely wiping the surface over with a moderately wet cloth) will achieve much (80 to 90 percent) of the total skid resistance reduction upon wetting; the addition of more water will lead to further slight reductions until a water depth of 0.5 to 1.0 mm (depending on surface texture) is reached. The addition of more water will result in a slight gradual increase in skid resistance, at least at higher vehicle speeds.

PRESENT TRENDS

An attempt is made in this section of the paper to outline the current trends in skid resistance research, both in North America and in Europe. While the picture presented is certainly not a comprehensive one, it is intended to be representative as far as the main avenues of progress are concerned. Some new developments, some recent arguments in areas of dispute, and some specific current research projects are mentioned.

For the sake of uniformity in organization, the material is presented under the same headings as in the preceding section: nature of skid resistance, measurement of skid resistance, and skid resistance parameters. A fourth heading is added to cover the utilization of research findings in measures taken to prevent skidding accidents.

Nature of Skid Resistance

Research is continuing on the basic theoretical concepts of tire-pavement friction, with particular reference to the distinction between the adhesion component and the deformation component of the frictional force. It is being emphasized (20, 21, 25) that the relative magnitude of these components depends not only on the conditions of lubrication but also, and to a great extent, on the properties of the pavement surface. There are "adhesion-producing" surfaces with very many and extremely fine asperities, and "hysteresis-producing" surfaces with much fewer, large, and rounded asperities. The adhesion term has been found (20) to be very much material-dependent and directly related to the damping properties of the tire rubber.

There is a laboratory study under way (36) concerning the thickness of liquid films trapped between rubber and a hard surface during sliding. While this is not specifically a skid resistance investigation, it is part of a broader attack on friction and lubrication. The immediate goal is to discover under what conditions lubrication effectively cancels the adhesion component of the frictional force.

A long-overdue new development is the current drive to create a standard terminology to cover the study of tire-pavement friction. It is hard to think of another field of engineering activities where the basic terms are so poorly defined, and so recklessly used, as in skid resistance research. A sampling of overlapping terms in the conventional terminology in the field includes skid resistance and skidding resistance, braking traction and braking friction, road friction and tire friction, sliding friction and locked-wheel friction, incipient friction and impending friction, gripping coefficient and break-away coefficient, braking-force coefficient and sideway-force coefficient, critical coefficient and peak coefficient. The establishment of uniform terminology would lead to discipline of language and clarity of concepts.

The main elements of a recently suggested (21) system of terminology are the following. "Skid" or "skidding" is defined as an uncontrolled motion of a vehicle and also, in accordance with general practice, the sliding or locked-wheel mode of operation of skid testers. The corresponding frictional resistance is referred to as "skid resistance." The expression "skid number" is introduced and defined as the ratio of skid resistance to wheel load, times 100. A tire is said to be "slipping" when its angular velocity is between that of a rolling tire and zero. ("Slip" is defined as the ratio of the difference between the angular velocities of the rolling and slipping tires to the angular velocity of the rolling tire, times 100.) In analogy to skid resistance and skid number the terms "slip resistance" and "slip number" are introduced. To identify the mode of slip more precisely, distinction is made between "brake-slip resistance," "cornering-slip resistance," and "drive-slip resistance."

Measurement of Skid Resistance

Correlation Studies—In order to derive common benefits from the individual research efforts, there has been a pressing need in recent years to correlate, if possible, the test results obtained with the different skid resistance testers.

In the United States, the first large-scale and well-organized comparison study was conducted in Virginia, just prior to the First International Skid Prevention Conference, in 1958 (8). Ten skid-test devices were compared, including six braking force trailers, two stopping distance test cars, a test car with a decelerometer, and a portable single-wheel type apparatus. The tests were performed on wet pavements at four test sites ranging from very slippery to good. The scatter of results obtained with the different devices was highly significant, varying from 0.20 to 0.40. Reproducibility appeared to be best for the stopping distance tests, worst for the drawbar-force type trailers.

Following this test series a considerable amount of development work was devoted to improving the skid trailers, particularly the calibration techniques and the force-measuring systems. The benefits became apparent during the second, more comprehensive Virginia correlation study conducted at Tappahannock, Virginia, in 1962.

This study (9) involved eight skid trailers, eight stopping distance test vehicles, and fourteen portable testers. The tests were performed on five specially prepared wet pavement surfaces, ranging in skid resistance from extremely low to relatively high. This time the locked-wheel trailers were found to give reliable and closely similar measurements, even the machines with different force-measuring systems. However, the scatter of the results obtained with the different trailers was still significant, and was apparently affected by both testing speed and surface characteristics. Very satisfactory correlation was established between the trailer test and stopping distance test results. Considering the test vehicles and the portable testers, the differences were large, especially at low friction values.

In Europe, a good example of such studies is the test series conducted in Paris (6) that compares the trailer method, the sideway force method, and the decelerometer technique. The three sets of test results appeared to be in good agreement. Less satisfactory results were obtained in a test series in Germany (27, 35, 39), which involved three of the most popular European test vehicles.

Several current research projects in Europe are aimed at establishing a meaningful relationship between the full-scale test vehicles and the portable testers (23).

Trends of Standardization—One direct result of the various correlation studies has been the full realization that the results of skid resistance tests are always performance (and not absolute) values, referring to a specific testing equipment and a specific set of testing conditions. This, in turn, has led to a growing desire on the part of all interested parties to establish a standardized testing technique.

In the United States, the locked-wheel skid trailer is on its way to becoming the standard equipment for routine field testing. This apparatus is simple and straightforward. It duplicates the first phase of the majority of skidding accidents, i.e., a locked-wheel skid. The interaction of a large number of factors in pavement-tire friction being as complicated as it is, it appears logical to use a method of measurement which eliminates extrapolation for such factors as speed, wheel load, tire size, and tread composition. Naturally, it is implied that the test trailer is constructed to closely simulate actual passenger car operation in all these respects.

It is expected that the standard skid resistance test will call for a locked-wheel skid trailer, sliding on wet pavement at a speed of 40 mph, mounting an ASTM standard skid test tire with a wheel load of 1085 lb (at rest). The ASTM standard tire (1) has an oil-extended SBR tread, its physical properties are closely specified, and it has a plain ribbed tread design. A tentative ASTM standard has been published (2) containing guidelines for constructing skid trailers.

The standardization process has been somewhat slower in Europe. The significant exception is Germany, where for some years now a locked-wheel trailer (Stuttgart-trailer) has been accepted as the standard testing apparatus (5, 27, 41). A trend has been noted in Europe (27) toward favoring smooth tires instead of patterned tires for skid resistance tests. The reason for this trend is that large and unpredictable variations in test results have been attributed to tread damage during locked-wheel tests and during continuous measurement of sideway-force or impending coefficients. Patterned tires are more susceptible to uneven wear and tread damage than smooth tires.

Other Developments—The requirements of modern road traffic as well as those of airplane operating conditions have led to the development of specialized test vehicles.

A skid trailer developed by the General Motors Corporation can perform skid tests at speeds around 90 mph. A trailer is being developed in Germany (5) to be used at testing speeds over 120 kmph. In Britain, a special test vehicle with a weight of 11 tons is being used on the Crowthorne research track of the Road Research Laboratory (14, 15) to investigate the effect of heavy wheel loads (up to 4.5 tons) and high inflation pressures (up to 300 psi). The variation of tire frictional forces with the amount of longitudinal and transverse slip is also being studied, by means of a vehicle carrying a test wheel whose longitudinal slip and slip angle are controllable. The skid trailer of the U. S. National Aeronautics and Space Administration (8) has been built to measure impending friction, which is the operative value with aircraft due to the elimination of wheel locking by automatic braking devices. Percentage slip may be varied from 0 to 50 percent on a very well instrumented Swedish test vehicle (17). A trailer-type device is being developed by the U. S. Federal Aviation Agency (37) which will measure friction coefficients on airfields under simulated landing conditions.

Skid Resistance Parameters

Pavement Parameters—Research concerning the effect of aggregate characteristics on the frictional properties of pavements is continuing along two main routes: full-scale tests on experimental road sections, and closely controlled laboratory tests.

The experimental road sections are laid by varying aggregate type, mix composition, and construction methods according to a predetermined pattern in segments along the test road. Traffic conditions are either controlled or closely observed, and periodic skid resistance measurements are taken. These studies normally take several years to complete but yield valuable data on a variety of pavement parameters and on the role of traffic and weathering. Such studies are either under way or are being planned in Tennessee, Texas, Florida, and New York. A study of this type was part of the AASHO Road Test (22), and similar investigations have been reported from Britain (14, 15), and Germany (5, 41).

Typical of current laboratory investigations are projects at the Swedish Road Research Institute (23), the National Crushed Stone Association, and the Portland Cement Association; these studies are aimed at determining the relationship between the petrographic characteristics of aggregates and their polishing susceptibility. The advantage of laboratory environment is that the interaction of the various skid resistance parameters can be simplified, or even the effect of one particular parameter can be isolated. However, the difficulty of simulating traffic wear and polishing must be faced. Several new accelerated-wear techniques are under development (23, 39).

Work is also continuing in connection with the other main pavement parameter, surface texture. The immediate task appears to be to express the texture of a pavement in quantitative terms, replacing the usual and somewhat ambiguous descriptors such as coarse, rough, and harsh. Present work by the National Aeronautics and Space Administration involves measuring the area over which a known volume of grease can be spread to just fill all surface depressions. A simple apparatus called the Outflow Meter, which provides quantitative measurements of the drainage capacity of pavement surfaces, has recently been constructed at the Pennsylvania State University (25). At the Crowthorne research track of the British Road Research Laboratory, measurements of surface texture are being made by a new stereoscopic method (15). Some preliminary work has been carried out (31) to establish the possible correlation between light-reflecting properties of a pavement and its frictional properties.

An extensive testing program is being conducted in Sweden (30) regarding skid resistance on ice- and snow-covered roads.

Vehicle Parameters—Considerable attention is being devoted to the role of rubber properties in tire friction, both in Europe (27, 36) and the United States (20). Recent developments include the significant finding (20) that high damping rubbers develop high adhesion as well as high hysteresis losses.

A study is in progress at the Crowthorne research station in Britain (14, 15) to evaluate the effect of the individual tread pattern elements, such as number of ribs, rib width, and rib separation. The behavior of different tires under different conditions of rolling and sliding is being investigated by photographing the contact patches under tires through an optical system built into the track. Significant reductions in skid resistance have been attributed to tire wear (10).

A relatively new area of research is concerned with the significance of studded tires in skid resistance and pavement wear. The use of studded tires on ice- or snow-covered pavements originated in the Scandinavian countries and has now spread to other countries in Europe and North America. There are now over 200 million studs manufactured in the United States per year (4). According to a recent test series (28), studded tires give about 40 percent higher effective skid resistance on ice than standard tires; the improvement is about 9 percent on packed snow, but is negligible on bare pavements. The studs appear to cause pressure spots, scratches, and grooves on the pavement surface, eventually leading to rutting (40).

Operating Conditions—The variation of skid resistance with temperature, seasons, and climatic conditions is also being further studied (23, 32). According to recent research (20), an increase in temperature leads to higher adhesion and lower hysteresis losses. The relationship between temperature and skid resistance can therefore be expected to depend on the relative significance of adhesion and hysteresis. It is a direct relationship on "adhesion-producing" surfaces, and an inverse relationship on "hysteresis-producing" surfaces.

Investigations are under way concerning frictional resistance at high vehicle speeds. It has recently been reported (14, 37) that coarse-textured surfaces are not only retaining a greater portion of their skid resistance at high speeds than the fine-textured surfaces, but they also exhibit a sharp recovery of skid resistance at speeds around 100 mph. The explanation has been offered (20) that the frequency required for maximum hysteresis loss on these surfaces is reached only at such high speeds.

The variation of skid resistance with speed is being examined in the light of surface texture parameters (18). The effect of the thickness of the water layer on the pavement and the hydraulics of run-off from road surfaces is also being studied (15, 23, 38).

Utilization of Research Results

Skid Resistance Standards—A common goal of many investigations, involving the consideration of stopping distances and the correlation of skidding accident data with skid resistance measurements, has been the establishment of minimum acceptable levels, or practical standards, of skid resistance. While most highway agencies now have such empirical standards, these are normally regarded as general guidelines only and not as legally binding standards.

The following standards have been suggested in Britain (13), in terms of sideway force coefficients measured at 30 mph: a minimum value of 0.60 for "difficult sites" (such as roundabouts, sharp curves, and steep gradients), minimum 0.50 for "general conditions," and minimum 0.40 for "easy sites" (straight and level sections without intersections). These values are based on measurements obtained with smooth tires. The possibility is now being explored (15) that measurements obtained with modern passenger-car tires may yield more reliable criteria.

For bituminous pavements in the city of Paris, a minimum sideway force coefficient of 0.50 is specified, measured one year after construction, at a speed of 30 kmph (30). In Belgium (30) a minimum sideway force coefficient of 0.50, as measured at 50 kmph, is required up to two years after construction. German specifications (5) call for a minimum sliding coefficient (skid-trailer) of 0.45 at 60 kmph.

Suggested minimum standards in the United States include 0.40 (26) or 0.50 (11) with locked-wheel trailers at 40 mph, 0.40 with the stopping distance method at 40 mph (29), and 0.40 with the decelerometer technique at 40 mph (24).

Anti-Skid Roads—In the final analysis the total research effort that has been discussed in this paper has just one single purpose: to develop ways and means of avoiding skidding accidents. One aspect of the problem is to make the roads safe by constructing surfaces which offer high frictional resistance throughout their service life. Failing in this high ideal, appropriate measures must be taken to correct existing slippery conditions.

In general terms, the requirements of adequate skid resistance call for angular and non-polishing aggregates, a harsh surface texture permitting fast drainage, and proper mix design (e.g., bitumen content) and construction methods (e.g., surface finish of portland cement concrete pavements). The best experiences in non-skid road construction (3, 33) have been obtained with silica-sand mixes, Kentucky rock asphalt, and slag mixes on the bituminous side, and with similar non-polishing aggregates and broom or burlap finish on the portland cement concrete side.

Slippery stretches of road are normally resurfaced using some non-skid mix. Special techniques include the application of resinous surface treatments (27), the removal of excess asphalt by burning, the dissolution of excess cement by acid treatment, and the mechanical roughening of portland cement concrete pavements.

Other Anti-Skid Measures—The other main aspect of providing adequate skid resistance on the roads is to increase the anti-skid characteristics of the vehicle itself. The two main possibilities are (a) improving the frictional properties of the tires and (b) controlling the braking action.

Tire friction can be improved by synthetic rubber tread compounds exhibiting high hysteresis losses and by bold tread patterns. Studded tires or tire chains may be needed on icy surfaces.

In order to utilize the maximum frictional resistance of pavements (impending friction or maximum slip resistance) devices are being developed (14, 16) which prevent wheel-locking during braking. Experimental versions of such devices have led to skid resistance improvements on the order of 10 to 30 percent.

Finally, the driver himself must, of course, be alerted to the dangers of skidding. He, too, can take anti-skid measures by adjusting his speed to road and weather conditions, and by using the "pumping" braking technique on wet or icy pavements.

FUTURE NEEDS

There is a definite need for continued research in many aspects of tire friction. Past results will have to be checked in the light of new developments and by means of the improved testing methods. Better understanding will have to be sought in areas of uncertainty or dispute. Some of the major research needs are summarized in this section of the paper.

A uniform terminology must be established so that the basic concepts of tire friction can be uniformly interpreted. It is strongly recommended that the HRB Subcommittee on Surface Slipperiness adopt such a standard, and insist on its use in any future reports.

In developing a better understanding of the basic mechanics of tire-pavement friction, a topic of high priority is the relative importance of the adhesion and deformation terms under different pavement, tire, and environmental conditions. Any further progress on this topic will automatically become the epicenter of new developments in the whole problem area.

The time is appropriate for international standards on measuring tire friction. Such a development would greatly enhance the benefits of individual research efforts; discussions and potentially fruitful arguments would not become bogged down on the relatively trivial point of how to measure the property in question. There would exist a common denominator to which points of common interest could be referred. Perhaps even more significantly, all routine tests could be performed by means of the standard method, the results of which would be easy to interpret in light of the widespread experience behind it.

The ASTM Committee on Skid Resistance is currently developing a standard method of measuring tire friction. It is the consensus of the membership of that committee that a skid-trailer would best satisfy the requirements for such a test method. At the same time, it is recognized that efforts should be made to further improve the trailer technique, particularly in the areas of trace evaluation, calibration procedures, speed recording, and brake control (19).

While the establishment of a standard approach is an immediate necessity, the use of nonstandard devices is not to be discouraged. On the contrary, there must remain a variety of measuring techniques in use, corresponding to the variety of goals of specific research projects. For example, further attention should be devoted to the relationship between tire friction and percentage slip; there are indications that on some pavements at high speeds tire friction increases with decreasing rotational speed, attaining a maximum at 100 percent slip (locked-wheel condition). It should also be possible to establish a meaningful correlation between locked-wheel coefficients and sideway force coefficients.

Of course, interest must be maintained with regard to new techniques of measuring the frictional resistance of pavements, such as the light reflectivity technique. It should be possible to evaluate the frictional behavior of a road surface in terms of its physical properties without a mandatory reference to a tire or some other rubber object actually sliding over it. Conversely, it should be possible to evaluate the frictional behavior of a tire in terms of its physical properties without actually dragging it over a pavement.

There is continuing need for laboratory investigations and for the reliable accelerated simulation of wear and performance under traffic. Nevertheless, such tests can never replace "real-life" testing programs. There is a definite need for laying full-scale test sections with both established and experimental design and construction methods, and for studying the performance of these sections under modern traffic conditions. Such projects may be unique in duration and perhaps in cost, but they are also unique in benefits.

In routine road construction, improved geometric design and better engineered traffic control systems should reduce the need for the critical vehicle maneuvers of cornering and braking (18).

Among the parameters associated with the pavement surface, quantitative characteristics of aggregates and of surface texture should be correlated with frictional resistance. A variety of ways are available for testing aggregates; the task is to decide which ones, or the combination of which ones, are relevant. Satisfactory and widely accepted techniques of quantitatively expressing surface texture should be developed. The effect of temperature, climatic factors, and traffic wear should be evaluated in terms of pavement type. It should be possible to predict with reasonable accuracy the future "skid resistance behavior" of a pavement at the time of construction, based on pavement and traffic parameters.

Among the parameters associated with the vehicle, basic rubber properties, tread pattern, tread depth, tire wear, wheel load, and suspension characteristics certainly require continued attention. The effect of vehicle speed should be expressed in quantitative terms. With regard to hydroplaning, it appears to be equally important to further explore the problem and to disseminate the knowledge presently available.

There is a pressing need for the establishment of realistic minimum acceptable values of frictional resistance in terms of pavement type, layout features, and traffic conditions (18). Such a scale can then be used to judge the adequacy, with respect to frictional resistance, of individual road sections. Similarly, criteria must be developed by which the adequacy of vehicle tires and braking systems may be judged, because all the present and future know-how on tire friction will have to be translated into eliminating skidding accidents by building safe roads and operating safe vehicles.

To realize these objectives, the public must be made fully aware of the implications of vehicle skidding, highway designers and contractors must appreciate the fundamental facts of tire friction, highway agencies must put into effect long-range research programs, and the individual research efforts must all be parts of an organized attack on the problem. It is the task of the HRB Subcommittee on Surface Slipperiness, the ASTM Committee on Skid Resistance, and the PIARC Technical Committee on Slipperiness to spearhead and to organize efficiently these developments.

ACKNOWLEDGMENTS

The authors wish to thank the leading research workers on both sides of the Atlantic for contributing valuable data toward the preparation of this paper. Special thanks are due to Mr. H. W. Kummer and Professor W. E. Meyer for the critical review of the first section.

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State of the Art of Pavement Condition Evaluation

•PAVEMENT CONDITION is a subject of concern throughout the United States. Certainly many civil engineers including pavement designers and maintenance personnel are interested in the subject. But by far the largest interested group is composed of pavement users. Every user seems to rate pavement condition either consciously or unconsciously every time he rides in a motor vehicle or during the ground run of an airplane. There are a great many reasons for evaluating pavement conditions and even more ways of doing it. The names applied to the process are varied and many of the definitions are unclear. Terms like performance, serviceability index, condition survey, sufficiency rating, performance rating, and others are often bandied about by engineers and laymen alike. The definitions of such terms, however, are not precise and differ for the various interested parties.

In this paper every attempt will be made to be precise in use of terms and definitions. These will be chosen in an attempt to agree with predominant usage and will be given as precisely as possible in order to establish a springboard for future work in this area.

In the study of pavement evaluation, two major categories emerge. These are (a) serviceability-performance studies of functional behavior, and (b) mechanistic evaluation for structural adequacy. Regardless of the method used to make the evaluation, most studies can be listed in one of these two main categories. In general, serviceability-performance studies concern themselves primarily with the overall behavior of the pavement, that is, how well it is performing its function as a riding surface for vehicular traffic. By and large this also seems to be the area of major concern to the user.

On the other hand, the mechanistic evaluation for structural adequacy of pavements with a view to current load capacity and future performance is a vitally important area of interest to pavement engineers, pavement designers, and maintenance engineers. The understanding of the interrelationship between these two categories is of interest and importance to us.

Pavement condition is studied for several reasons. A few of these are:

1. To furnish information needed for sufficiency ratings and needs studies. This involves a comprehensive study of pavement systems within an area such as a state.
2. To aid the design engineer in the determination of the degree of success with which his design has met the design criteria and to help him learn causes for failure.
3. To aid in the establishment of priority for major maintenance, reconstruction and relocation. The object of this type of survey is to rank various pavement sections in terms of their importance and their current ability to serve traffic.
4. To assist the maintenance engineer and administrator in the determination of an optimum maintenance program.
5. To assist in the determination of the load-carrying capacity of the pavement both as to volume of traffic and loads. This involves an evaluation of structural adequacy of the pavement structure, climatic effects, materials, and drainage.
6. To serve as a basis for new concepts and design.

SERVICEABILITY-PERFORMANCE STUDIES

Serviceability—Subjective Rating

The evaluation of pavement performance involves a study of the functional behavior of a stretch of pavement in its entirety. For functional behavior or performance analysis, information is needed on the trend of the effect of load applications on the ability of the entire pavement to serve traffic. It can be determined by periodic observations and measurements of the pavement surface coupled with records of traffic history. Such studies are extremely important in the evaluation of pavement design.

Up until the time that the present serviceability index (6) was developed in conjunction with the AASHO Road Test, little attention was paid to evaluation of pavement performance per se. A pavement was either satisfactory or in need of repair; the ideas of relative performance were not adequately developed. Most pavement design concepts in general use did not consider the level of performance desired. Design engineers as a group vary widely in their concepts of desirable performance. As an example, suppose that two designers are asked to design a pavement for a certain traffic environment for 20 years. The first might consider his job to be properly done only if not a single crack occurred in twenty years, whereas the second might be satisfied if the last truck which was able to get safely over the pavement made its trip at the end of the twentieth year after construction.

Many popular design systems involve determination of the pavement thickness required to hold certain computed stresses below certain levels. It is clear that cracks will occur if the pavement is overstressed, but not much information was available prior to the time of the AASHO Road Test to relate such cracks to functional performance. The "pavement serviceability-performance concept" developed by Carey and Irick (6) for use at the AASHO Road Test is a well-defined technique for evaluating pavement performance.

Philosophy of Ratings— A rating implies the construction of some type of arbitrary scale to be used in the rating. Teachers often rate students on a scale of 100 percent; amateur golfers are rated by an arbitrary system called a handicap which is derived as a percentage of their average score over par for a period of time. Many such arbitrary scales in use today could be cited as examples. For many years the "roughness index" was used as a rating scale for pavements. This roughness index is rather arbitrary, and a "good value" depends largely on the particular piece of equipment used in the evaluation.

If some absolute roughness standard were available, this problem would be minimized. It is not likely, however, that such an absolute standard will ever be developed. As a result, "scaling factors" have been developed to provide a basis for comparing ratings from many sources throughout the world. Although many scales could have been chosen, a scale of 0-5 is in current use throughout the United States (5). Anyone rating a pavement is asked to scale his judgment from 0 to 5 using 5 as a possible perfect score.

PSI Developments—At the WASHO Road Test it proved to be especially difficult to establish a failure condition for pavements subjected to the test traffic. As a result of these difficulties the idea of average pavement ratings was conceived. As stated by Carey and Irick (6), there are five fundamental assumptions associated with the pavement serviceability concept. These may be summarized as follows:

1. Highways are for the comfort and convenience of the traveling public. Stated another way, "a good highway is one that is safe and smooth."
2. The user's opinion as to how he is being served by highways is on the whole subjective.
3. There are, however, characteristics of highways that can be measured objectively which, when properly weighed and combined, are in fact related to the user's subjective evaluation of the ability of the highway to serve him.
4. The serviceability of a given highway may be expressed by the mean evaluation given by all highway users. Honest differences of opinion preclude the use of a single opinion in establishing serviceability ratings. The mean evaluation of all users, however, should be a good measure of highway serviceability.
5. Performance is assumed to be an overall appraisal of the serviceability of a pavement. Thus it is assumed that the performance of a pavement can be described if one can observe its serviceability from the time it was built until the time its performance evaluation is desired.

Based on these fundamental assumptions, Carey and Irick developed the PSI system used at the AASHO Road Test. Their evaluation shows that pavement roughness or the pavement profile is closely related to pavement serviceability ratings. Furthermore, the AASHO Road Test (1) showed that performance measured in this manner is correlated with certain pavement design factors.

Human Sensibilities—Hutchinson (16) discusses some of the problems associated with subjective ratings. Care must be taken in the development of such rating systems and improved rating scales can no doubt be developed if additional attention is given to this subject.

The evaluation of riding quality is a complex problem, depending on three separate complex systems plus interactions between them: pavement user, vehicle, and pavement roughness. Hutchinson (14) has described the problems associated with analyzing the subjective experience of highway users in deriving an absolute measure of riding quality. These require (a) the development of a suitable mathematical model to characterize pavement roughness, (b) the development of a suitable mathematical model to describe the suspension characteristics of highway vehicles that may be used along with the roughness model to predict the dynamic response of vehicles, and (c) a quantitative knowledge of the response of humans to motion.

In order to improve our subjective rating systems it will be necessary to objectively evaluate human sensibilities including the effect of motion sickness and its causes. These no doubt will involve studies of frequency, wavelength, and amplitude.

Surface Evaluation

Present serviceability is largely a function of pavement roughness. Studies made at the AASHO Road Test (6) have shown that about 95 percent of the information about the serviceability of a pavement is contributed by the roughness of its surface profile. That is to say, the correlation coefficients in the present serviceability studies improved only about 5 percent when cracking and patching were added to the index equations. Hveem (17) discusses this problem in several papers. He states that "there is no doubt that mankind has long thought of road smoothness or roughness as being synonymous with pleasant or unpleasant." Road surface roughness is not easily described or defined, and the effects of a given degree of roughness naturally vary considerably with the speed and characteristics of the vehicle.

Roughness Defined—What is pavement roughness? It is a phenomenon produced by a pavement surface and experienced by the passenger and operator in a vehicle or airplane traveling over that surface. Pavement surface roughness is a function of the profile of the road surface, the parameters of the vehicle including tires, supension, body mounts, seats, etc., and the acceleration and speed sensibilities of the passenger. All of these factors undoubtedly affect the phenomenon of roughness. Safety considerations will also influence our acceptance of roughness. Most people refer to pavement roughness as "the distortion of the pavement surface which contributes to an undesirable or uncomfortable ride." This definition then refers to the pavement alone and divorces itself from subsequent considerations. The evaluation of pavement roughness by this definition cannot of course be made until a great deal more is known about true profile, vehicle dynamics, and human response. For the purposes of this report, however, this definition will suffice.

To completely define the roughness function some evaluation of the roughness of the entire area of the pavement should be made. However, for most purposes this roughness can be divided into three components: transverse variations, longitudinal variations, and horizontal variations of pavement alignment. In other words, any functional roadway which imparts acceleration to the vehicle or to the passenger must be examined. More particularly of interest are those functions which influence the comfort and safety of the passenger. There are many previous studies which have shown that longitudinal roughness is probably the major contributing factor to undesirable vehicle forces (6). The next greatest offender is transverse roughness (e. g., the roll component transmitted to the vehicle). The general curvature of the roadway which imparts yaw forces to the vehicle is considered to be the least offensive and one which is normally handled by following good highway alignment practices. Since most vehicles (approximately 70 percent) travel in a well-defined wheelpath with their right wheel

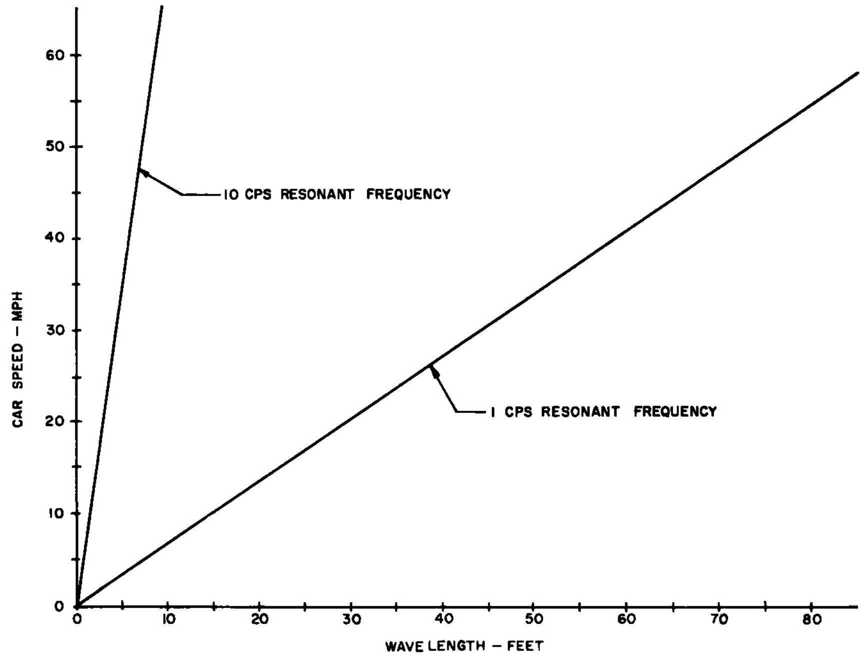

Figure 1. Relationship between wavelength, car speed, and car resonant frequency.

located $2\frac{1}{2}$ to $3\frac{1}{2}$ feet from the right-hand lane line, we are tempted to conclude that measurements of longitudinal profile in the two respective wheelpaths 6 feet apart might provide the best sampling of roadway surface roughness. Furthermore, comparison between the two wheelpaths can provide some measurement of the cross slope or transverse variations which are also important.

A passenger riding in a vehicle passing over a road surface experiences a ride sensation. This ride sensation is a function of the road profile, the vehicle parameters, and the vehicle speed. A variation of any one of these three variables can make a rough road appear smooth. Then we might say that from a vehicle passenger's viewpoint, roughness is an unfortunate combination of road profile, vehicle parameters and speed. Riding characteristics of airplanes are also affected by the properties of the

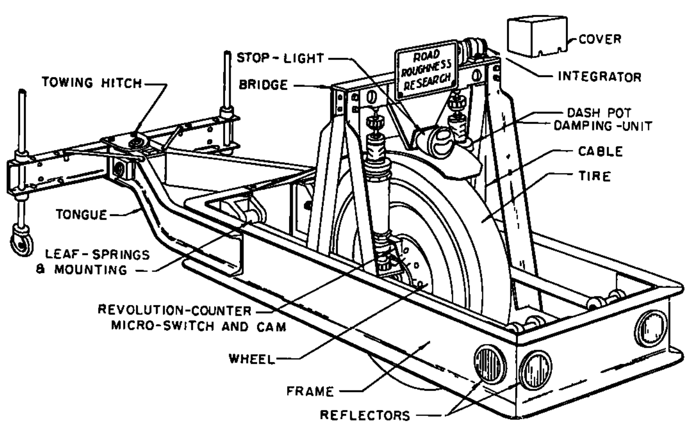

Figure 2. Bureau of Public Roads roughometer.

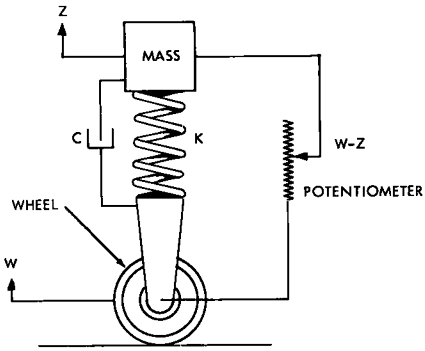

Figure 3. Mechanical vibrometer.

pavements and of the equipment. Accelerations of sufficient magnitudes to critically affect safety of operations are sometimes measured over poor pavements.

In general, most passenger-car ride characteristics are much alike, and the vehicle parameters (tires, suspension, body mounts, seats, etc.) are not changed sufficiently to make a significant change in passenger comfort. With the limitation of relatively fixed vehicle parameters it becomes apparent that ride sensation is almost completely a function of the car excitation generated by the various combinations of road profile and vehicle speed. Most drivers have experienced the sensation of either slowing down or speeding up to improve the ride on a particular road. This indicates that the road has a wavelength content that when driven over at some speed produces an excitation into the car at one of the car's resonant frequencies. The typical passenger car has resonant frequencies at approximately 1 and 10 cycles per second. The relationship between wavelength, car speed, and car resonant frequency is shown in Figure 1. This relationship indicates that at any speed there is a road wavelength that will cause an excitiation at one of the car resonant frequencies. If the amplitude of that wavelength is large, the car ride will be noticeably affected.

We have said that, in general, most passenger-car ride characteristics are very much alike. We can also say that for any particular road most cars will be driven at about the same speed. With two of these variables held relatively fixed, the excitations into the car and thus the riding characteristics of the car are strictly a function of the wavelength content of the road profile surface.

We have discussed the interrelationship between the roadprofile, vehicle parameters, and vehicle speed in producing a ride sensation. The final ingredient in the road roughness picture is passenger sensibility. So far, we do not know enough about the passenger to know what he or she objects to in the ride sensation, but we can feel sure it is related to the road profile and more directly to the wavelengths in the road that cause the car to resonate.

Roughness Equipment—Researchers in the highway roughness area have long realized that it is important to study the characteristics of the highway surface over which the car is driven. Hveem (17) in 1960 presented a good survey of early road surface measuring devices. Many of the devices mentioned in his paper are no longer being used. Most of the present-day research in road surface evaluation in the United States involves the use of one of the following devices: Bureau of Public Roads roughometer, rolling straightedge, slope measuring device, or GMR profilometer.

The Bureau of Public Roads roughometer (Fig. 2) is essentially a mass, spring, and damper combined to form a device called a mechanical vibrometer. These components are arranged as shown in Figure 3. In effect, the device is a simulation of one wheel of a passenger car. The displacement of the wheel with respect to the mass is measured as the device passes over the road surface at 20 mph. This displacement is accumulated over a distance interval and is called the roughness index with units of inches of displacement per mile. A transfer function for this device is shown in Figure 4. The roughometer reduces the amplitude of road wavelengths longer than 17 feet and shorter than 2 feet. Wavelengths of 3 feet are amplified by a factor of 10. Figure 5 compares the amplitude measured by the roughometer at 20 mph with the amplitudes felt by the car passenger at 60 mph. This figure shows that the roughometer amplifies the shorter wavelengths or wavelengths that cause car shake but attenuates wavelengths that are considered in the ride frequency range. As the car speed goes up it would appear that the roughometer measurements, which are made at 20 mph, would have less meaning.

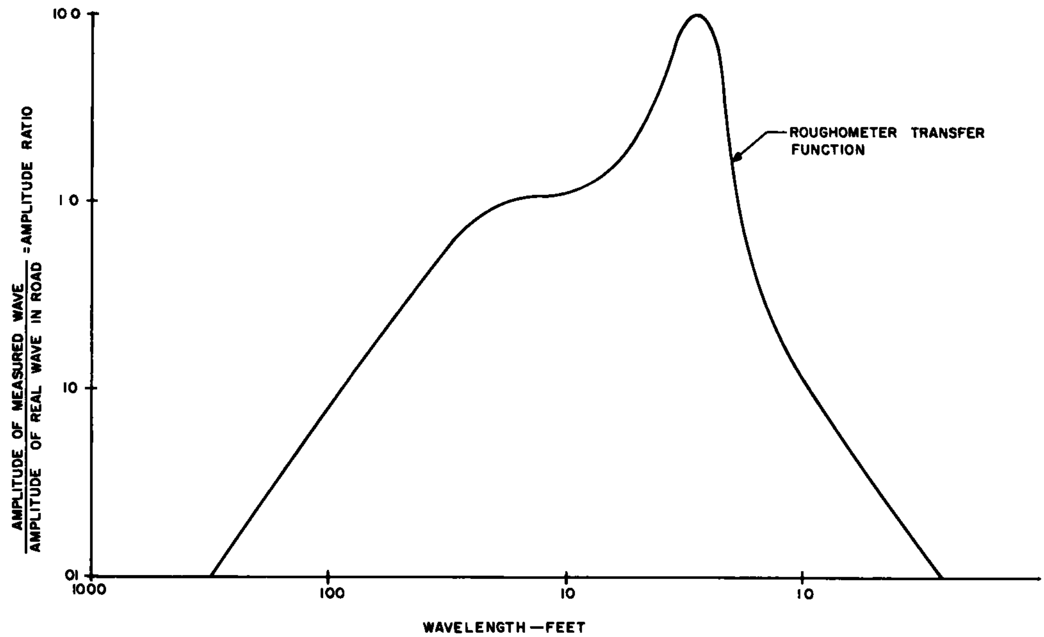

Figure 4. Roughometer transfer function.

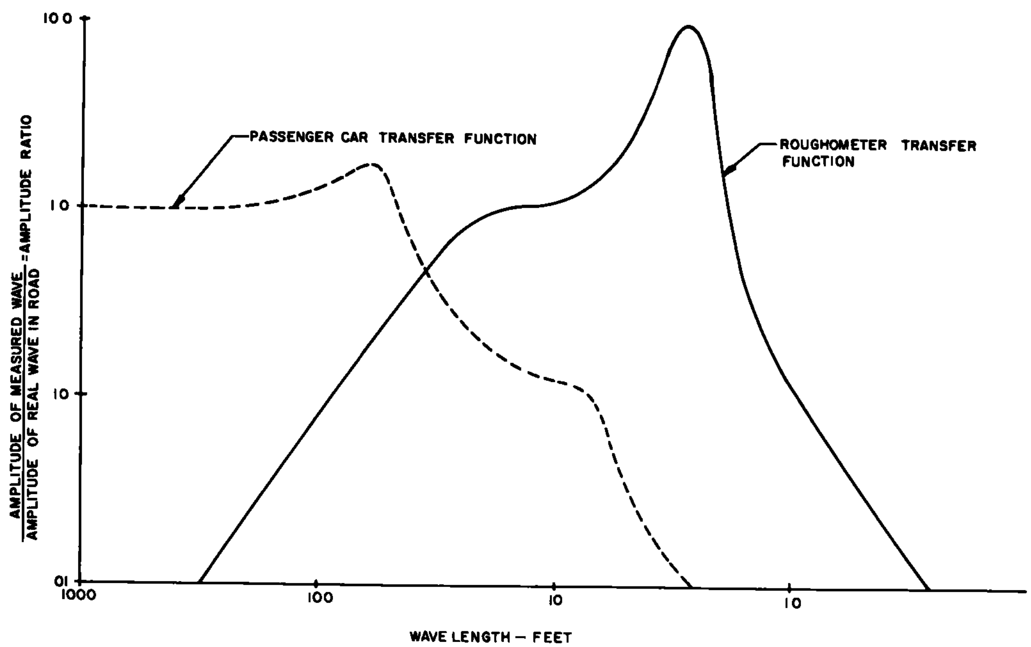

Figure 5. Comparison of amplitude measured by roughometer at 20 mph with amplitude felt by passenger at 60 mph.

Figure 6. Michigan profilograph.

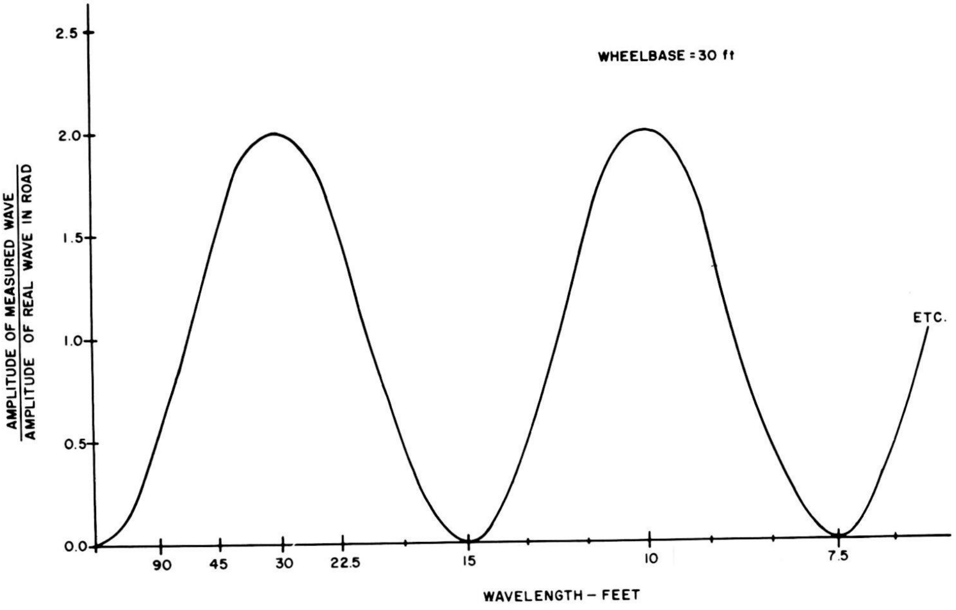

Figure 7. Rolling straightedge transfer function.

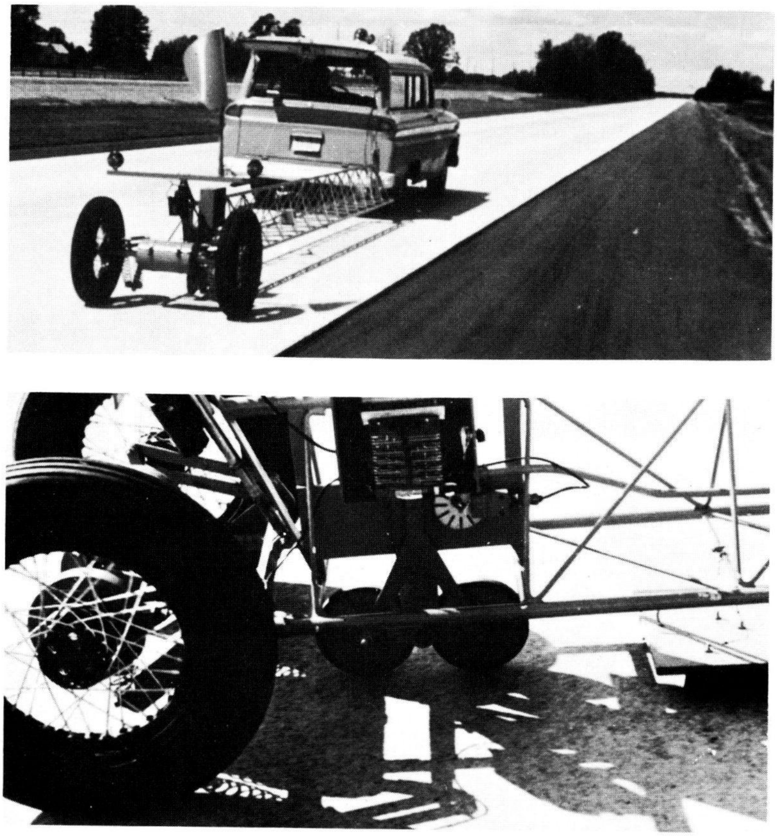

Figure 8. CHLOE profilometer.

The rolling straightedge (Fig. 6) is used by several groups in this country. Both the California Highway Department and the University of Michigan have truck versions of this device. It has been useful in extensive road condition studies at the University of Michigan. Figure 7 shows the transfer function of this system. The simple rolling straightedge has the serious disadvantage of badly distorting the wavelength content of the road profile it measures. Figure 7 shows that the rolling straightedge does not tend to respond to waves whose lengths are $\frac{1}{2}$, $\frac{1}{4}$, $\frac{1}{6}$, $\frac{1}{8}$, etc., of the overall wheelbase of the machine. Since the road wavelength information that is desired may fall in this area, it appears that this device would have limited usefulness in the evaluation of road roughness.

The CHLOE profilometer (Fig. 8) developed for use in the AASHO Road Test is a good example of the slope measuring vehicle. In this device the change in angle between two reference lines is the measure of the pavement profile roughness. One reference line is determined by two slope wheels which follow the road and are relatively close together. The second reference line is determined by a 20-ft long member which is supported by a trailer hitch on the back of a towing vehicle and a wheel which supports the rear end of the member.

A transfer function that relates the slope measured and actual slope is not available for the CHLOE profilometer. But considering the geometry of the device, it appears that wavelengths shorter than the distance between the two slope wheels will not be measured accurately. It also appears that information on the longer wavelengths will be lost completely. The determination of the transfer function for the CHLOE profilometer is complicated by the motions of the towing vehicle which must be included since it is also following the road profile.

Figure 9. General Motors profilometer.

The GMR road profilometer (Fig. 9) is a recent development in road surface measuring equipment (32). This device measures the profile of the road surface over which it passes. The wavelength content of the road profile is measured accurately from the very short waves to the longer waves (up to 400 ft). Figure 10 is the transfer function for the GMR profilometer for a measuring speed of 40 mph. Since this device measures all the wavelengths in the road that are important to vehicle ride, it appears that this device should be usable in future road roughness studies.

Of the four devices discussed for measuring road profile characteristics, the GMR road profilometer is the only device whose output contains information on all of the

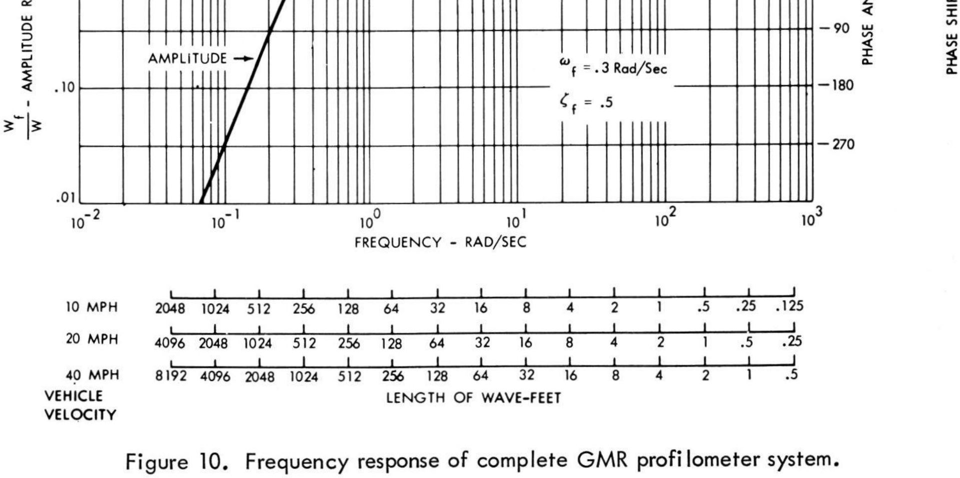

Figure 10. Frequency response of complete GMR profilometer system.

wavelengths important to vehicle ride. Accurate measurement of the road profile, however, does not tell the user of the device anything about the roughness or ride characteristics of the road. It is in this area that research activity is needed. Considering the system of the vehicle and human body riding in the vehicle, we have the ability to measure the input to the system, the road profile, and the output of the system—the passenger's opinion of the ride sensation.

However, the passenger's opinion is subjective and requires the use of a passenger or group of passengers to evaluate each road for roughness. Through future research it is hoped that an objective ride criterion can be formulated that will allow the completely objective evaluation of a road system. The complete evaluation procedure would consist of measuring the road profile, using the road profile as an input into a simulation of a typical vehicle, using the motions of the typical vehicle as an input into a simulation of a typical human body, and the monitoring of the outputs of the simulated body to determine its reaction to the road. It may be that the simulation of both the vehicle and human body can be reduced to an instrument box whose input would be the measured road profile and whose output would be a road roughness evaluation in the form of a number or a curve. It is obvious that there is much work between the present status and the desired future position. A major step has been taken in our newfound ability to measure and record road surface profiles. The simulation of a typical vehicle does not appear to present a problem but the final step of obtaining an objective passenger ride criterion will require an extensive amount of research. The sooner we start on this research the sooner we will have the ability to rate road roughness objectively.

While it is true that none of the existing roughness equipment is perfect, it can all be put to good use. Many agencies throughout the United States are making estimates of pavement serviceability and thence performance. They are using the roughometer, the CHLOE profilometer, and many other approximate devices. Until better equipment is developed, the continued use of existing methods is essential to pavement evaluation.

Needed Research—In order to make further advances in this area it will be essential that equipment for measuring "true profile" of the pavement surface be developed. These true profiles will make it possible to characterize pavements very accurately and in subsequent correlations with studies of human sensibilities should provide improved serviceability index equations. Additional research is also needed in human response to motion since psychologists have pointed out that it is very difficult to obtain realistic subjective ratings from human subjects. Operational and safety requirements of airplanes in relation to pavement roughness is also a subject which requires extensive research.

Condition Surveys

Condition surveys are important to the mechanistic evaluation of highway pavements. They will be thoroughly discussed in the next section. Studies at the AASHO Road Test (6) showed that condition surveys were also helpful in improving correlations in the present serviceability index. All the studies to date have shown that an evaluation of surface cracking and patching of all pavements and the addition of faulting measurements on portland cement concrete pavements will provide adequate information to improve correlation coefficients between pavement ratings and serviceability indexes by about 5 percent. More detailed condition surveys do not seem to be warranted for serviceability-performance studies.

Traffic History (Loads)

Among the many diverse and complex factors which affect pavement performance, the number and magnitude of loads has a very direct effect on road life. An accumulative record of serviceability ratings has also been shown to be a measure of the performance of a pavement (6, 1). Therefore, the relationship of accumulative serviceability ratings and number of axle loadings provides a means of evaluating the actual performance of the pavement to the expected or designed pavement life.

The importance of knowing the number and magnitude of axle loads for purposes of evaluating pavement performance therefore becomes apparent. The ideal situation of course would be axle loads of constant magnitude; thus procurement of data would be simply recording the number of axle load applications. Unfortunately, this is not the case for the mixed traffic which travels the nation's highways. It becomes necessary therefore to accumulate information on volumes of traffic for the pavements in question and to derive some relationship of number and magnitude of loadings for various classifications and volumes of mixed traffic.

The AASHO, Maryland, and WASHO Road Tests (1, 20, 35) have shown that the frequency of application of heavy loads directly affects the service lives of highways. Results of these complete studies provide a means of relating the destructive power of one wheel loading to another; for example, two applications of a 30,000-pound axle loading is considered equivalent to six applications of an 18,000-pound axle load. The number and rate of application of heavy axle loads therefore is an important aspect of highway evaluation. Several test tracks built and tested by the U. S. Army Corps of Engineers for the U. S. Air Force verified the same concept and established the relationship between airplane gear loads, land applications, and airfield pavement designs. These relationships were later extrapolated for the design and evaluation of military roads and streets.

Most states are making studies of the number and magnitude of axle loads, but it is not possible to obtain such information continuously for all highways and streets. Therefore, the available data must be expanded and assumptions made to apply the information acquired to a statewide basis.

To establish an approximation of number and magnitude of loads for a particular pavement, it is first necessary to determine the volume and classification of the traffic. Traffic volumes generally follow cyclic variations such as the season of the year, the day of the week, and the hour of the day. Also, it is known that traffic volume has been increasing and is expected to continue. Traffic volume is influenced by many factors, such as national economy and international relations, and therefore is difficult to predict with any degree of accuracy. Past experience has shown that predictions of increases have generally been conservative.

The highway traffic stream is composed of passenger cars, light trucks, medium trucks, heavy trucks, and buses, and the amount of each is subject to variations due to type and location of the road or land use along the road. The percentage of total number of vehicles that each of the various types of vehicles represent also varies by time of day and week. Periodic measurements of traffic movement therefore are necessary to maintain as reliable a basis as possible for predictions.

To complete the accumulation of necessary information, the vehicles are weighed by various means. The methods used in obtaining the weights of these vehicles vary from weighing the vehicle while moving slowly over a scale to use of small portable scales which measure the load on each wheel. Many states operate permanent scales at strategic locations on primary highways on a periodic or continuing basis, and the majority of the weight information comes from these stations. Generally, all trucks are required to cross permanent scales during the period of operation, whereas random sampling of vehicles is used for the portable scales.

Since total numbers of axles and axle loads are extrapolated from these samplings of vehicle weights, it is important that the samples be reliable. In a report by Shook et al (31), it was pointed out that the reliability of the samples is governed by such factors as:

1. Size of sample: (a) percent of the daily total count which is weighted; (b) number of stations at which surveys are made; (c) number of stations included for each type of highway; and (d) number of observations made each year at each station for each highway type.
2. Method by which specific vehicles are selected.
3. Time of day and year during which weighings are made.
4. The specific locations of the weighing stations (a) relative to the character of the traffic, and (b) relative to the type of highway they represent.

Shook (31) also points out that axle load and truck type distributions varied not only from state to state but within a state for different types of highways. Therefore, extrapolation of data from one location to another or to a relatively large area requires recognition of the assumptions which are applied and the errors which may be introduced.

Continual load studies by the states will result in more accurate estimates of the loading history of the particular section of pavement being evaluated. However, the ultimate goal will be a continuing record of actual magnitude and number of loads being applied. Norman and Hopkins (25) reported on an electronic weighing device which measured axle weights, axle spacings, and speeds of vehicles moving at their normal speeds along highways. Electronic and structural problems developed during the study, but the potential of such a scale was demonstrated. More recently a report by the Kentucky Department of Highways (19) describes the construction, installation, testing and performance analysis of three types of dynamic electronic scales: the Taller-Cooper, a commercially developed four load cell scale; the broken bridge, an adaptation of a German prototype employing two load cells; and the beam-type scale, an experimental prototype that uses a pair of instrumented aluminum beams as the weight sensors. The report concludes that all three scales will accurately measure the applied load, but that the broken bridge and beam scales appeared to be more suited for collection of data for use in pavement design and highway planning. The Taller-Cooper scale appeared to be better suited to research in pavement and vehicle dynamics. All three scales were equally suited for collection of statistical axle load data and enforcement of axle weight limitations. The report also includes an excellent bibliography with synopses of some of the more pertinent entries.

It is readily apparent from the available information that highway agencies are currently expending considerable research effort in the field of traffic loadings and that methods of obtaining a continuous record of axle loadings at a particular site without interfering with traffic are being developed.

Performance

A definition is needed for the term "performance." Several definitions have been proposed in recent years. In general, these definitions agree that the "performance" of a pavement is the "ability to serve traffic safely over a period of time." Webster defines performance as "the execution of the functions required of one; often, effective operation, as of a motor." Applied to pavements then, the term pavement performance means "the effective operation of the roadway in its function of carrying traffic." Carey and Irick (6) define performance as the "trend of serviceability index with time." They define performance index as "a summary of PSI values over a period of time."

Since the AASHO Road Test and the use of the serviceability-performance concept in analysis of the road test data, considerable misunderstanding of the basic concept has been demonstrated. No one ever intended a single PSI value to be a measure of pavement performance. Just as the runs scored in any particular inning of a baseball game do not indicate the final outcome of the game, the PSI does not indicate the performance of a pavement, nor was it intended to. However, just as the accumulation of runs throughout the course of a baseball game ultimately adds up to the final score, the accumulation or total evaluation of the serviceability history of a pavement can be evaluated to measure final performance of the pavement.

MECHANISTIC EVALUATION FOR STRUCTURAL ADEQUACY

Condition Surveys

Although it may not be a matter of record, one can state with a fair degree of certainty that condition surveys must have developed about the same time in history as the turning wheel. Once the advantages of the wheeled vehicle became apparent, the road builder or the highway engineer certainly was needed.

As roads developed it was, no doubt, a keenly observant individual who first saw the advantages of using strong granular materials over the less stable natural fine-grained soils for building roads. His knowledge was not based on elaborately equipped laboratories and libraries but came through an understanding of what was available for him to see, that is, through observations of pavement condition. Since that time the engineers and pavements have progressed a long way, but it is still important today to evaluate the various elements that make up today's pavement on the basis of actual field performance. Condition surveys provide the necessary information to compare the role played by each element in the overall performance of the pavement. The designer, the builder, the user, and the maintenance engineer all have an important stake in pavement performance.

The full impact of the use of the serviceability index to rate pavements will possibly not be fully realized for many years. Certainly we should seek ways to improve and extend its usefulness and to develop a better system. It is not, however, intended (nor likely) to do away with the making of condition surveys, which is one of the most basic tools for extending our knowledge of highway engineering.

It is recognized that a series of PSI values obtained on a particular section of pavement over a period of time, when correlated with traffic histories and environment, is an indicator of design, materials, construction, and maintenance variables that exist. However, conditions seldom prevail except on special test projects where there is not a strong influence of each of these factors on the performance of the pavement. Yet, since not all pavements have the same capacity to perform, it becomes necessary that critical inspections be made by knowledgeable personnel to establish the cause, or causes, for the variation in performance. Pavements often fail to perform satisfactorily for a combination of reasons. These are the difficult ones, and often no single solution is easily obtained. In many other cases they have been correctly analyzed, and additional information is made available. Thus, each one of our thousands of miles of pavements serving under a great variety of traffic and environmental conditions serves as one more element in a vast proving ground.

Condition surveys made to establish the structural adequacy of a pavement usually are made in more detail than is normally required for establishing the PSI of a pavement. They generally include not only a record of all locations or the number of times a particular kind of distress is observed, but also indicate the degree to which the distress has developed, such as class 1, 2, or 3 cracks. Types and condition of maintenance operations are also important data.

Most pavement engineers have at one time or another been involved in making condition surveys, or at least have been exposed to reports made on the basis of information obtained from them. There appears to be no single method of making a condition survey that is used universally. Because of the many uses made of this information, an extremely wide variation exists in the manner in which the surveys are obtained, recorded, analyzed, summarized, and stored. Each perhaps has its special advantages and/or disadvantages. It is not within the scope of this report to list or judge their merits. It does seem important, however, that a list of standard definitions of items included on condition surveys be agreed upon and used. In most respects this has been done and has been reported in HRB Special Report 30 (26), which also contains a variety of suggested forms that can be used. Some examples of reports prepared from condition surveys are given elsewhere (2, 7, 12, 22, 27, 28, 29), and there are others available in the literature.

Improvements in the methods and techniques used in obtaining data from condition surveys are slow to develop. One area that is presently receiving some attention is the retrieval of construction information. In some cases this information is being placed on IBM cards, which should cut down the time in the office needed to dig this information out. In addition to readily supplying the information on a specific project, this method is also extremely useful in helping to select the proper sections to survey.

Another area in which there has been some relatively new developments is the use of pictures or strip maps made by special cameras mounted on a truck. We should encourage the development of any idea which would tend to reduce the time required, improve the accuracy or cut down on the cost of condition surveys.

Nondestructive Tests

An evaluation of the structural adequacy of the various components of an existing pavement without disturbing or destroying these components is highly desirable. To accomplish this, measurements must be obtained on or above the surface of the pavement and the results related to the structural properties of the underlying elements. Measurements of responses of a pavement structure to an external force or energy are referred to as "nondestructive" since the structure of the pavement is not altered and such measurements can be repeated at the same location. Nondestructive testing methods can be separated into three general categories: measurements of response to a selected static load or a single application of a slow-moving load, response to a repeated load, and response of a mass to a controlled source of nuclear energy.

The response to a single application of load is generally obtained by measuring the deflection of the pavement surface. Pavement deflection under a wheel load is usually measured by means of a Benkelman beam. The Benkelman beam was developed at the WASHO Road Test (35); it is a portable instrument which produces measurements of deflection to a thousandth of an inch. Results of a study in California (18) indicated that when surface deflections of flexible pavements as measured by the Benkelman beam exceeded a certain value, the subject pavements generally showed signs of distress. A similar study in Virginia (23) resulted in the same general conclusions. Comparison of surface deflections to a critical deflection value, therefore, provides a means to program maintenance for flexible pavements. Studies at the AASHO Road Test (1) indicated that relations existed between surface deflections and performance of flexible pavements; thus surface deflections can also be used as a means of evaluating pavement performance. The Benkelman beam is a simple instrument to operate, but variables such as temperature of the pavement and curvature of the deflection basin (9, 10) require careful consideration when interpreting the results.

Plate bearing tests have also been used by agencies to obtain deflections of pavement under load. The Portland Cement Association (38) has developed and used a method to determine values of modulus of subgrade reaction of underlying layers by plate loading of rigid pavements and measuring strains at the surface of the pavement as well as deflections.

Deflections of surfaces under repeated moving loads have been measured by means of linear variable transformers installed within a pavement structure. Considerable information on this method has been published (1). Although nondestructive, the method does require a permanent installation at one point in a pavement. The influence of such an installation, which is foreign to the surrounding media, raises the question of the effect on the results.

A series of vibration measurements was conducted on flexible pavements at the AASHO Road Test and the results reported by Nijboer and Metcalf (24). Initially the procedure consisted of exerting an alternating vertical force on the surface of the pavement and measuring the deflection of the surface or the velocity of wave propagation.

Measurement of the surface deflection provides an elastic stiffness value for the total structure being loaded whereas the wave velocity values can be interpreted to determine the stiffness of the various layers. Heukelom and Klomp (11) have reported on such measurements for soils and stabilized and unstabilized base courses in various European locations. These reports by Nijboer and Metcalf (24) and Heukelom and Klomp (11) provide extensive bibliographies.

Vibratory equipment was used by the U. S. Army Engineer Waterways Experiment Station (33) to determine the elastic modulus of soils under pavements. The method used was basically that developed by the Shell Oil laboratory in Amsterdam, Holland, which consists of setting up a steady state of vibrations at a given frequency and measuring the velocity of the propagated waves. This is essentially the same method as mentioned in the preceding paragraph.

The Experiment Station used an empirically developed half-wavelength procedure for interpretation of velocity. By using the E modulus developed by these techniques and resorting to the elastic theory, computations were made to determine pavement strengths. Although relative strengths of pavements could be obtained by these methods, it was questioned that the procedures were developed to the point where pavement strength could be accurately evaluated.

A report by Scrivner and Moore (30) describes a study conducted in Texas using a dynamic loading system and measuring surface deflections by means of geophones placed in contact with the surface. Deflections of the surface produced by the dynamic loading are compared to Benkelman beam deflections under a single load application. Results of the study indicate that a relatively good correlation existed between the two methods of determining pavement deflections. The dynamic deflection equipment is quite rugged and can be operated in the field by one man. The mobility of the dynamic equipment and the short time required for actual testing are favorable factors to consider.

At the present time, nuclear testing provides measures of density and moisture content of pavement materials (3). Nuclear equipment has been used experimentally for determination of asphalt content of bituminous mixtures (34) as well as compacted density of hot asphalt pavement (3). The normal use to date of nuclear equipment is in maintaining control of construction procedures, although it is conceivable that application may develop toward evaluation of constructed pavements as well. One example is measurement of changes in density of a base course subjected to traffic for a year before the surface material is placed. A limited program of this type was recently conducted in Wisconsin.

At the present time the nondestructive methods of testing, briefly described above, provide good indications of the structural adequacy of the pavement material itself and that of the underlying layers. Certainly none of these methods can be considered as producing accurate measures of the strength properties of the underlying layers. Recent advances in the field of electronics and nuclear detection may yield new methods of nondestructive testing that will provide more accurate measurements of the structural capacity of the various components. However, considerable research and development is necessary before such methods become available to highway agencies.

Destructive Testing

Although the performance of pavements can be evaluated by measurements of surface irregularities or the logging of pavement defects such as cracking and rutting, it becomes necessary occasionally to remove portions of the pavement structure to ascertain just where the failures are occurring and why. The term "destructive testing" is applied to these evaluation methods since the original structure of the complete pavement is destroyed with respect to future testing at that particular location. In general, such evaluation procedures are restricted to pavements that show evidence of distress; however, they have been used on test roads (1, 20, 35) to determine the evolution of distress.

The techniques used depend on the type of information desired, but generally involve cutting into each pavement layer and removing samples for testing. At times the objective is to obtain undisturbed samples of the various layers. However, the successful attainment of this objective may not always be realized due to the circumstances involved.

The actual cross section of the various layers of rutted flexible pavements can be studied to analyze the behavior of each layer and the functioning of the system. One such study in Kentucky (8) revealed that subgrade soil had intruded into the waterbound base course material, thus suggesting changes in the gradation of the base course material and modifications of certain construction procedures.

Trenches were cut transversely across flexible pavements at the AASHO Road Test to obtain information concerning the amount of wheelpath rutting at the top of each of the component structure layers as well as to obtain information on the existing condition and strength of the materials. It was found that rutting of the pavement was due principally to decreases in thickness of the component layers attributed to lateral movement of the materials. These results along with density and strength tests on samples of the removed material provided considerable information on the structural capabilities of the pavement.

Several states are currently conducting research on degradation of base course materials after subjection to service under traffic. Samples are removed from the base course layer at various intervals of time and tested in the laboratory to determine what increase, if any, in fines has occurred. Removing the existing surface to allow sampling of the base material is undesirable; however, the information gained by sampling and testing material exposed to actual service conditions counterbalances this detrimental aspect.

The authors believe that many states excavate and examine isolated trouble spots in pavements to determine the cause of the particular problem and take steps to correct the situation. These individual investigations are rarely reported in publications; in fact, the information rarely goes beyond the individual group involved in the actual problem. Consequently, the available information concerning destructive testing methods and the attendant results is limited to those occasions where these methods were incorporated into an overall program of evaluation such as at Road Tests.

The advantages of opening up pavements for detailed investigations below the surface must be weighed against the disadvantages of removing portions of the pavement and replacing with patches. It is important that all variables that affect pavement performance be evaluated before definite conclusions are reached. Too great a reliance on the appearance of defects at the surface should not be made, for many times this may give misleading results. Surface defects can be used as general guides to the underlying conditions; however, it is often necessary to determine the true position and cause of failure for a completely reliable analysis.

COMPARISON OF FUNCTIONAL EVALUATION VS MECHANISTIC EVALUATION

Pavement condition can clearly be analyzed from two different points of view. The first of these embodies a study of the functional behavior of a stretch of pavement in its entirety, while the second is a study of the mechanics of pavement behavior at specific locations. Many names have been applied but, based on the statement of committee activities published by Highway Research Board Committee D-B5 (Pavement Condition Evaluation), we have referred to these two points of view as a functional evaluation and a mechanistic evaluation. There is some honest difference of opinion and considerable misunderstanding between these two evaluation techniques. Much of the misunderstanding seems to arise from engineers who have used one or the other of the methods of evaluation extensively, but have never used the other method and therefore are not familiar with it.

Much of the misunderstanding comes from the ingrained feeling among engineers with a background of structural experience that a crack in a structural unit designed by engineers is an indication of failure. In some instances cracking is synonymous with failure; yet such is not the case with all engineering structures and certainly not in the case of pavements. For example, many properly designed prestressed concrete beams continue to function well and carry their designed loads for many years after cracks appear in the concrete itself. As another example, continuously reinforced concrete pavements function well with cracks. Many designers (21) feel that they function better with fairly close crack spacing, thus improving "performance." The pavements are in fact designed to crack at these spacings rather than at longer spacings.

A crack per se may or may not affect the function of a pavement. In some cases, certainly continuously reinforced pavements as cited above, cracking is not detrimental and may be helpful. Data cited by Carey and Irick (1) in developing PSI concepts show that pavement raters pay scant attention to cracks. A rough crack (spalled or faulted) will, however, add roughness to the longitudinal profile and will result in a higher roughness measurement, and thus a lower serviceability index. It can be seen that it is not the crack itself but more particularly the condition or roughness of the crack that affects the function of a pavement.

On the other hand certain cracks, no matter how fine, may be indicators of structural inadequacy to engineers of trained judgment. This depends on the type of crack and its cause. Mechanistic evaluation of pavements is involved with the investigation of such cracks and other pavement deterioration and specifically with the determination of the causes.

It should be reiterated in this summary that pavement performance cannot be predicted from a single PSI value. Trends of the PSI or serviceability history are required, and thus some loss in serviceability must be observed and some mathematical model must be employed to make life or performance predictions. Some engineers today are doing this by using the Road Test equation as a mathematical model and estimates of the initial or starting serviceability of the pavement sections being evaluated. Such efforts may be helpful in predicting average or "possible" pavement life. But such predictions can be misleading and have in the past given some users the idea that pavement performance was being predicted from a single PSI determination. Such is not the case.

Determination of failure mechanism is difficult even though some important work has been done in this area. Ex post facto observations are usually confounded by rapid destruction of pavements near failure, the difficulty and expense of so-called destructive sampling, and the fact that undisturbed samples are very hard to obtain. Furthermore, there are indications that failure mechanisms exist on a microscopic scale whereas sampling and testing procedures take place on the larger macroscopic scale.

SUMMARY

Pavement condition has been judged for centuries, but until recently these judgments have been subjective and qualitative instead of objective and quantitative. Functional observations, for example, involve statements such as "this is a good road," "poor road," "best road," "worst road," etc. Pavement engineers have likewise made mechanistic evaluations of almost every road ever built. These have varied in approach in detail and in results gained. However, much of what we know about pavements has come from such observations. Early test roads and experimental pavement sections relied heavily on such evaluations and the interpretation of such results. Many mechanistic evaluations were made at the AASHO Road Test (1) and were helpful in determining mathematical models and other phases of data analysis.

The establishment of a failure criterion is essential for all test sections and tracks such as the AASHO Road Test. The PSI or Present Serviceability Index is the result. The history of PSI with traffic or axle application is termed "performance." The clarification and use of such a system as the serviceability-performance system is essential in any Road Test satellite program or any nationwide study of pavement performance. Only through such common denominator factors can the multitude of variables across the nation be compared.

Present Practice

At the present time a good many states are observing functional behavior of highway pavements. Many are using PSI determinations as evidenced by the ownership of 17 CHLOE profilometers and 25 roughometers plus various other devices in current use. These functional evaluations are being put to various uses, but many of them are involved in the nationwide Road Test satellite program in an attempt to better define factors affecting pavement design and performance.

Mechanistic evaluation of pavement conditions is also continuing. Nondestructive tests are becoming more and more important in such mechanistic studies as the problems and expenses associated with destructive testing techniques increase. The difficulties involved with digging test pits or making other destructive tests in the main lanes of an interstate highway make the use of nondestructive tests more and more desirable. Such studies of mechanistic failures and search for possible causes will continue to be an important aspect of pavement condition evaluation.

Future of Pavement Condition Evaluation

The future of pavement condition evaluation will undoubtedly lead to solutions of many of the current problems facing pavement researchers. Research problem statements submitted by the HRB Pavement Condition Evaluation Committee include the following items (the statements are numbered for convenience; no attempt has been made to list them in order of priority or importance):

Problem No. 1: To develop a more rapid and reliable procedure for evaluating pavement condition. The objective of this project is to produce a method for evaluatin pavements which eliminates the need for annual measurements for such defects as cracking and patching. Such a procedure would make it possible for a great many mo pavements to be evaluated than is now possible for most highway departments. This would results in a corresponding increase in the usefulness of such data in decision-making processes.

Problem No. 2: To devise improved control techniques for pavement smoothness during construction. The objective of this research would be to develop better method of specifying and controlling pavement smoothness during construction in order to establish construction control specifications for pavement quality.

Problem No. 3: To develop evaluation techniques for determining the load-carrying capacity of existing pavements and thus the needs for preventive maintenance. The objective of this research is to seek better methods for predicting future serviceabilit and thus for predicting load-carrying capacity of existing pavements.

Problem No. 4: To establish a psychologically based subjective rating scale for use in determining the relative riding quality of a pavement. The objective of this research is to establish a more realistic scale for pavement rating based on recently developed information. Such a scale should account for "lenient errors," "central tendency effect," and "halo effects" that are normally present in subjective ratings performance by human beings.

Problem No. 5: To clarify the serviceability performance concept. The objectives of this research would be (a) to clarify the pavement serviceability concept, (b) to develop the best way for evaluating serviceability as a method of determining performance of pavements, and (c) to differentiate between highway sufficiency ratings and serviceability ratings.

Problem No. 6: To determine the effects of environment and time variations on roughness equipment. The objectives of this research are (a) to determine the effect of environment, particularly temperature and humidity, on the operating characteristics of roughness-measuring devices used to measure pavement serviceability; (b) to collect available data necessary to establish control charts pertinent to the behavio of the various kinds of roughness equipment in current use; and (c) to evaluate the causes and effects of instrument variations throughout their operating life that may appear to be variations in pavement serviceability.

Problem No. 7: To determine factors in the pavement profile that affect passenger ratings of pavement serviceability. The objectives of this research are (a) to evaluat human response in an effort to determine the factors in riding quality which most influence subjective rating of the ride, (b) to make a detailed analysis of pavement profiles in an effort to break them into many components which are found to influence the subjective rating given by automobile occupants, and (c) to combine the evaluations in (a) and (b) to develop a riding quality evaluation which will more accurately predict the rider's acceptance of the quality of the ride and hence the present serviceability.

The accomplishment of the research set out plus many other factors which need studying will ultimately lead to improved methods of evaluating pavement condition. For functional evaluations these must lead to better equipment, better rating methods, more knowledge of pavement profiles, vehicle characteristics and the effects of motion on the human mind and body. For mechanistic evaluation these studies must lead to a more thorough knowledge of the mechanics of pavement load-carrying capabilities and pavement failure, better knowledge of the strength and physical properties of the various components of the pavements, and better methods of determining the strength and physical properties of these pavement layers nondestructively.

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THE NATIONAL ACADEMY OF SCIENCES is a private, honorary organization of more than 700 scientists and engineers elected on the basis of outstanding contributions to knowledge. Established by a Congressional Act of Incorporation signed by Abraham Lincoln on March 3, 1863, and supported by private and public funds, the Academy works to further science and its use for the general welfare by bringing together the most qualified individuals to deal with scientific and technological problems of broad significance.

Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

The NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

The NATIONAL RESEARCH COUNCIL was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to enable the broad community of U.S. scientists and engineers to associate their efforts with the limited membership of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities.

Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

The DIVISION OF ENGINEERING is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

The HIGHWAY RESEARCH BOARD, an agency of the Division of Engineering, was established November 11, 1920, as a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the Board are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.

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