Of course there is nothing new in the use of reinforcement in highway construction and, in a sense, the membrane placed even upon the surface may be considered as a form of reinforcement. However, because of its complete continuity, its adaptability for use with flexible or non-rigid types of construction offers a comparatively new field of scientific research There is a need for accurate measurements of the effectiveness of a membrane placed upon or slightly below the surface of a flexible pavement; a study of the relative effectiveness of various combinations of materials in the formation of membranes; and particularly a study of the durability of different membranes under actual service conditions. If membranes are found to be economically effective in highway and runway construction their use will result in a decrease in the required thickness of the ordinary types of base and subbase construction but not as a substitute for such construction.

THE RATIONAL DESIGN OF BITUMINOUS PAVING MIXTURES

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SYNOPSIS

The use of the triaxial test and the Mohr diagram for designing the strength or stability of bituminous mixtures on a pounds per square inch basis is outlined in quantitative terms A method for calculating the amount of lateral support provided by the pavement material adjacent to the loaded area is included. The influence of the viscous resistance of bituminous mixtures on their stability is described The effect of the frictional resistance between pavement and tire and between pavement and base on the stability of bituminous pavements is discussed The influence of braking stresses on the design of bituminous mixtures is considered. Stability equations and design charts are included

An engineer becoming acquainted for the first time with current engineering practice in the field of pavement design for airports and highways, must be surprised to learn that our approach to the design of rigid pavements is entirely rational throughout, that is, on a pounds per square inch basis, while flexible pavement design is largely empirical.

For rigid pavement design, the thickness of slab required can be obtained from the Westergaard equations. In addition, there are well established principles for designing portland cement concrete mixtures of any specified compressive or flexural strength in terms of pounds per square inch.

For flexible pavement design, on the other hand, there is the greatest divergence of opinion concerning the overall thickness of base and surface required, and the method to be employed for determining it That this problem of the overall thickness of flexible pavements is highly controversial at the present time, is quite evident from articles and discussions on this topic that have appeared in the technical press in recent years.

When it comes to designing bituminous mixtures of any specified strength, the stability tests in most common use, Hubbard-Field, Marshall, and Hveem Stabilometer, are not able to measure the strength of bituminous mixtures in terms of shear or any other fundamental property on a pounds per square inch basis. They are strictly empirical tests.

Empirical methods have a serious drawback in that it is dangerous to extrapolate their results to cover conditions beyond those under which they were established With empirical methods also, it is difficult to avoid either overdesign or underdesign In addition, in every engineering field there should be the ultimate objective of establishing rational methods of design, in which the strengths of all materials employed are utilized on a unit strength basis

In the absence of a more fundamental approach, these empirical tests have served usefully in the past to provide some indication of the relative stabilities of bituminous mixtures However, the fact that the number of such methods continues to increase is proof of the current dissatisfaction of highway and airport engineers with the inadequacies of these empirical tests. In addition to the three methods already named, the Texas Punching Shear, Florida Bearing Value, Modified Hubbard Field, Campen's Bearing Index, Unconfined Compression, Beam, Impact, Tensile, and Indentation Tests, etc., have been developed in recent years. This search for a satisfactory stability test is not likely to end until a generally acceptable fundamental approach to the measurement of the stability of bituminous mixtures is developed. This will probably require a considerable period, and a great deal of discussion

The Canadian Department of Transport has been interested in the development of a rational method of design for bituminous paving mixtures, partly because of the confusion that exists at the present time concerning the actual significance of the various tests such as Hubbard-Field, Marshall, Hveem Stabilometer, etc., employed to measure their stability, and partly because it has been suggested that large future aircraft may be

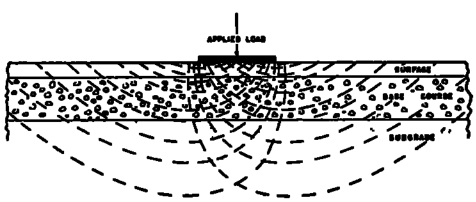

Figure 1. Diagram of Shear Planes under a Loaded Area

equipped with landing wheels carrying tire pressures of 300 to 400 psi The Department of Transport would like to determine whether or not dense, durable, bituminous mixtures can be designed to carry heavy wheel loads under such high tire pressures

Past experience has indicated how bituminous mixtures must be designed to have adequate workability, density, durability, etc

This paper is confined to the description of a method for measuring the strength of bituminous mixtures on a pounds per square inch basis, after they have met the general requirements in these other respects, and to a discussion of its utilization for the design of bituminous pavements.

Of all the methods of test presently available for this purpose, the triaxial test seems to be the most promising An outline of the manner in which the data provided by this test can apparently be utilized to design bituminous paving mixtures of any required stability on a unit strength basis, is the principal objective of this paper.

Any reference to the triaxial test immediately calls to mind the names of a number of investigators on this continent, who have made important contributions to our understanding of this test, Endersby (*1*)[1], Housel (*2*), Hveem (*3*), Smith (*4*), Terzaghi (*5*), Casagrande (*6*), Taylor (*7*), Rutledge (*8*), Holtz (*9*), and others.

Nijboer (*10*) and Smith (*4*) have developed methods for utilizing data from the triaxial test for the design of bituminous paving mixtures on a unit strength basis. Nijboer makes use of the Prandtl equation for this purpose. Smith's development is derived from the mathematical theory of elasticity, and is, therefore, subject to whatever uncertainties may result from the application of this theory to stressed materials close to the loaded area.

Before proceeding to a discussion of the triaxial test and its application to bituminous pavement design, brief mention should be made of the three principal conditions of pavement stability that must be considered. These are:

1. Stability under stationary loads.
2. Stability under loads moving at a relatively high and reasonably uniform rate of speed
3. Stability under the braking and accelerating stresses of traffic

When pavements are subjected to two or to all three of these types of load, it is necessary to determine which of the loading conditions is most severe from the point of view of pavement stability The stability of any bituminous mixture should be designed for the most critical condition of load to which it is likely to be exposed for a period of time during its useful life

THE TRIAXIAL TEST

The general nature of the problem can be more easily visualized by reference to Figure 1, which is a diagram of possible surfaces of shearing failure under a loaded area on a flexible pavement on an airport or highway. The overall design problem consists of preventing detrimental shear within any one of the three elements of the composite structure, the subgrade, the base course, and the wearing surface If sufficient plastic shear develops in any one or more of these three elements,

[1] Italicized figures in parentheses refer to the list of references at the end of the paper.

rutting and upheaval of the pavement surface will occur.

Detrimental plastic shear of the subgrade is prevented by an adequate overall thickness of base course and wearing surface (*11, 12, 13*). Serious plastic shear of the base course and bituminous pavement can be avoided, only if the materials selected for these two layers have adequate shearing resistance to the stresses of the applied loads.

For this paper, it is assumed that an adequate thickness of base and surface have been provided to protect the subgrade, and that the base course material itself will not fail under the shear stresses imposed by the loads

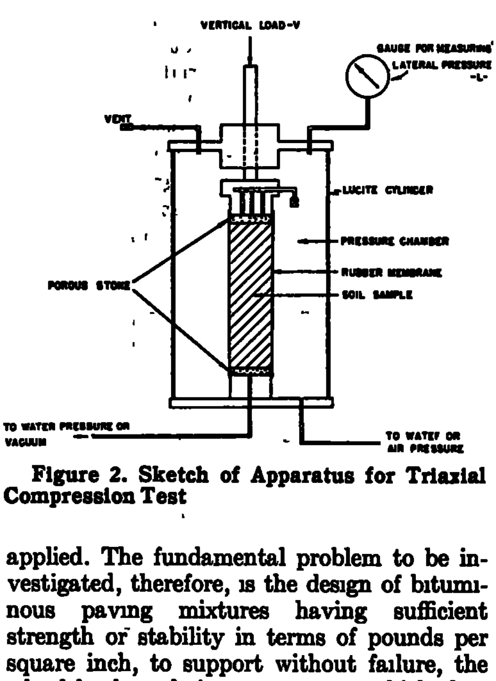

Figure 2. Sketch of Apparatus for Triaxial Compression Test

applied. The fundamental problem to be investigated, therefore, is the design of bituminous paving mixtures having sufficient strength or stability in terms of pounds per square inch, to support without failure, the wheel loads and tire pressures to which they are to be subjected. The development that follows attempts to provide a rational answer to this problem on the basis of information provided by the triaxial test and the Mohr diagram.

The triaxial differs from an ordinary compression test in that provision is made for the application of controlled or measured lateral support to the specimen while it is being subjected to vertical load. The triaxial equipment most commonly used on this continent is illustrated in Figure 2 It is sometimes referred to as the open-type, because the lateral support is maintained constant throughout the test on any one specimen. The two metal end pieces are fitted to the lucite cylinder by means of water-tight and air-tight gasketed joints. A cylindrical specimen of the material to be tested is inserted in a rubber sleeve. Porous stones at the top and bottom of the specimen may or may not be required, depending upon the material to be tested, and the nature of the test data desired By means of connections through the porous stones, the specimen within the rubber sleeve can be subjected to vacuum or water pressure, or to free drainage or not drainage, as required. Air, water, or other fluid can be pumped into the lucite cylinder to provide the magnitude of lateral support specified for the testing of each specimen. The rubber sleeve prevents the fluid within the lucite cylinder from entering the sample under test. Each specimen is subjected

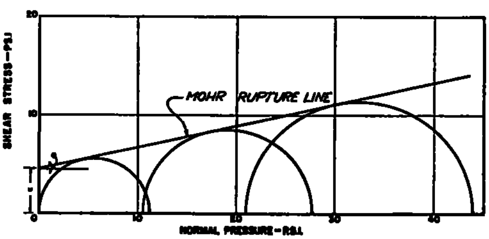

Figure 3. Typical Mohr Diagram for Triaxial Compression Test—$\phi = 12°18'$, $c = 4.8$ psi.

to a constant lateral pressure throughout the test, and increasing vertical load is applied in a standard manner until it fails. A complete triaxial test on a given material with this apparatus usually consists of loading three or four cylindrical specimens of the material to failure, employing a different degree of lateral support for each, e.g. 0, 15, 30, and 60 psi.

The data obtained from testing a given material in triaxial compression are plotted in the form of a Mohr diagram, Figure 3. For each specimen, the applied lateral pressure *L*, and the corresponding vertical pressure *V* that caused failure, are marked off on the horizontal axis, (abscissa). Using the difference between the vertical and lateral pressure, *V-L*, for each specimen, as the diameter, semicircles, known as Mohr circles, are described as shown The tangent common to the Mohr circles is drawn and produced to intersect the vertical axis (ordinate) The intercept on the vertical axis is designated cohesion *c*, while the angle between the common tangent and the horizontal is the angle of internal friction ϕ. Both c and ϕ are from the Coulomb equation, $s = c + n \tan \phi$.

The common tangent is generally known as the Mohr rupture line, or Mohr envelope. Mohr envelopes for different materials have a wide range of values for both c and ϕ

Figure 4 indicates that all semi-circles that are tangent to or below the Mohr envelope for a given material represent equilibrium

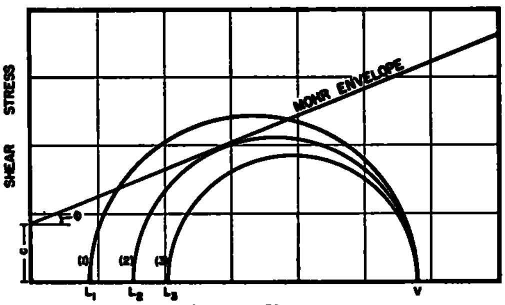

Figure 4. Mohr Circles representing Unstable, Equilibrium, and Stable Combinations of V and L Values for a Given Material under Stress

(circle 2) or stable (circle 3) relationships respectively between corresponding values of lateral support L and vertical pressure V. Any semi-circle (circle 1), which cuts through the Mohr envelope indicates corresponding combinations of lateral support L and vertical pressure V that would cause failure of this material.

For the development which follows, it is assumed that the Mohr envelope is a straight line. Whether or not this assumption is justified in the case of materials like clay soils, seems to depend upon the conditions of testing and the method employed for interpreting the results (*9*). It has been reasonably well established for purely granular soils (*8*), and the work of Nijboer (*10*) and Smith (*4*) indicates that a straight line Mohr envelope can ordinarily be expected for properly designed bituminous mixtures

Depending upon the position of the Mohr envelope that results from testing them, cohesive and granular materials can be conventionally divided into three groups.

1. Purely cohesive materials, i e those for which the angle of internal friction ϕ is zero, but the cohesion c has a positive value, and the Mohr envelope therefore, is parallel to the abscissa (Fig 5). Saturated clays in the quick triaxial test approximate these requirements (*8*), and bituminous mixtures with voids approximately filled or overfilled with bituminous binder, are probably other examples

2 Purely granular materials, i e those for which the cohesion c is zero, but the angle of internal friction ϕ has a positive value, and the Mohr envelope passes through the origin (Fig. 6) The clean sands, gravels, crushed stone and similar granular materials employed as the aggregates for bituminous mixtures approach these requirements

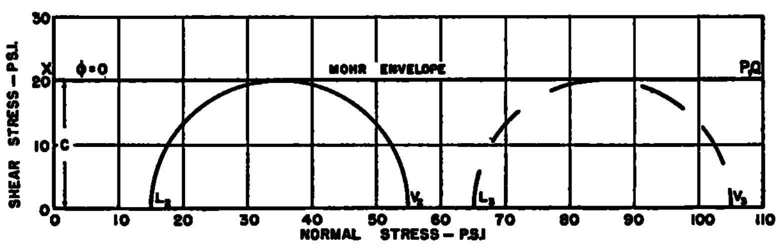

Figure 5. Mohr Diagram for Materials having Zero Angle of Internal Friction in Triaxial Compression

3. Materials which have both granular and cohesive properties, i e those with positive values for both cohesion c and angle of interval friction ϕ, and with Mohr envelopes of positive slope and making positive intercepts with the ordinate axis (Fig. 3) Bituminous paving mixtures (*10*) and remolded clays (*9*). are examples of materials with this type of Mohr envelope.

The Mohr diagram provides a fundamental basis for defining the term "stability" as applied to granular and cohesive materials in general, and to bituminous mixtures in particular. If several different bituminous mixtures were formed under standard conditions into cylinders of the same size (e.g. 6-in. dia. by 12 in. high), the same magnitude of lateral support L provided for each cylinder, and the vertical load V (applied under standard conditions) at which each cylinder failed was determined, it would be generally agreed that the most stable mixture was the one that carried the greatest vertical load V at failure. Consequently, for any specified value of lateral support L, the most stable material is that for which the value of V-L is the greatest at failure That is, the stability of a material under load is measured by the quantity V-L, where L is the amount of lateral support provided and V is the maximum vertical load it can carry without failure

This definition of stability is illustrated by Figure 7, in which Mohr envelopes ab, cd, and ef are given for three different materials At lateral support L, it is apparent that the material represented by Mohr envelope ef is the most stable of the three, since V_3-L_1 is greater than V_2-L_1 for Mohr envelope cd, and than V_1-L_1 for Mohr envelope ab. On the other hand, at lateral support L_2, the stability rating of the three materials is exactly reversed, with the material represented by Mohr envelope ab being the most stable, since V_6-L_2 is greater than V_5-L_2 for Mohr envelope cd, and than V_4-L_2 for Mohr envelope ef. Figure 7, therefore, serves to emphasize the fact that the stability rating, V-L, for each of a group of materials, depends upon the magnitude of the lateral support L at which the stability determinations are made.

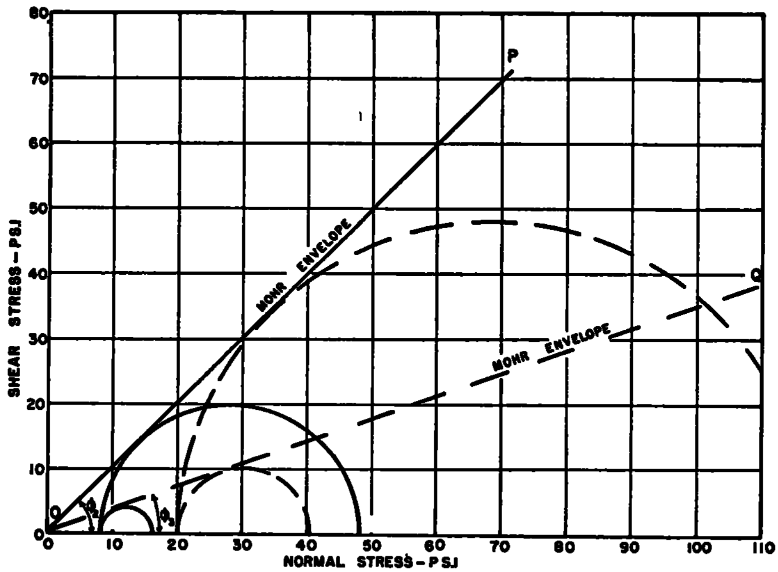

Figure 6. Mohr Diagram for Materials having Zero Cohesion in Triaxial Compression

DERIVATION AND APPLICATION OF AN EQUATION OF STABILITY

Figure 8 illustrates the geometrical and trigonometrical relationships required for the development of an equation of stability for materials with both granular and cohesive properties. From Figure 8 it is clear that the magnitude of the stability value, V-L, depends upon the magnitude of the lateral support L, the cohesion c, and the angle of internal friction ϕ.

From Figure 8 it follows that

$$\tan\phi = \frac{\frac{V-L}{2}\cos\phi - c}{L + \frac{V-L}{2} - \frac{V-L}{2}\sin\phi} \quad (1)$$

which can be easily worked through to

$$V - L = \frac{2L\sin\phi}{1-\sin\phi} + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}} \quad (2)$$

Equation (2) is an equation of stability for materials with both granular and cohesive properties

The stability diagram of Figure 9 is obtained when equation (2) is plotted in terms of given values of stability V-L, for different degrees of lateral support L, and for various magnitudes of c and ϕ. Each stability curve, V-L, shown in Figure 9, indicates that only those materials with combinations of c and ϕ that lie on or to the right of the curve would have the stability required for the combination of vertical load V and lateral support L specified for that stability curve.

Equation (2) can be rearranged as:

$$V = L\frac{1 + \sin\phi}{1 - \sin\phi} + 2c\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}} \tag{3}$$

Equation (3) is also a stability equation for materials with both granular and cohesive

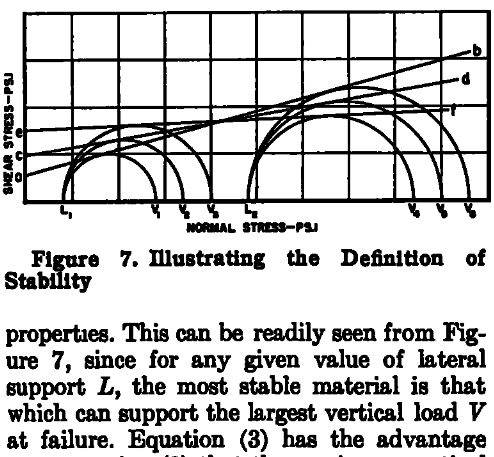

Figure 7. Illustrating the Definition of Stability

properties. This can be readily seen from Figure 7, since for any given value of lateral support L, the most stable material is that which can support the largest vertical load V at failure. Equation (3) has the advantage over equation (2) that the maximum vertical load V that can be supported by any given material is provided directly in terms L, c, and ϕ

The stability diagram of Figure 10 is obtained when equation (3) is plotted in terms of given values of vertical load V for different magnitudes of lateral support L and for various values of c and ϕ.

Figure 11 illustrates the practical application of equation (3) and Figure 10 to the solution of a given stability problem. If a material having both cohesive and granular properties is to carry a vertical load V of 100 psi. when the lateral support L is 30 psi. what values of cohesion c and angle of internal friction Φ are required? The graphical solution to this problem given in Figure 11 indicates the possibility of an infinite number of answers All materials possessing those combinations of c and ϕ, which are on or to the right of the curve labelled V = 100 psi., L = 30 psi , would have the required stability. Materials with combinations of c and Φ that lie within the cross-hatched area to the left of this line would tend to be unstable and therefore unsatisfactory insofar as this particular problem is concerned.

LATERAL SUPPORT PROVIDED BY PAVEMENT ADJACENT TO THE LOADED AREA

The general equations of stability (equations (2) and (3)) and the stability diagram of Figure 11 for a particular set of design requirements, V = 100 psi and L = 30 psi , while

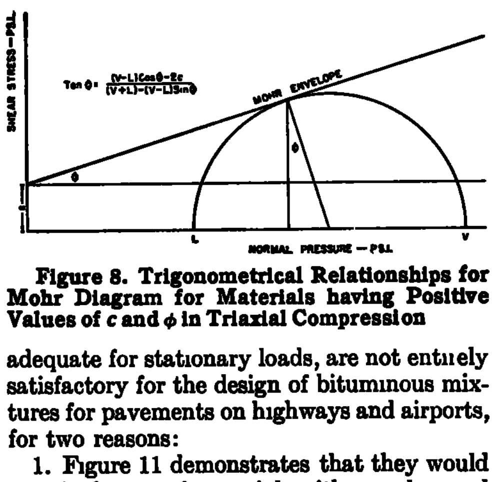

Figure 8. Trigonometrical Relationships for Mohr Diagram for Materials having Positive Values of c and ϕ in Triaxial Compression

adequate for stationary loads, are not entirely satisfactory for the design of bituminous mixtures for pavements on highways and airports, for two reasons:

1. Figure 11 demonstrates that they would permit the use of materials with very low and even zero cohesion c, since c becomes zero when ϕ is about 32.5 deg. Experience has indicated that only those materials containing sufficient binder to provide an appreciable value for cohesion c are capable of withstanding the particular types of stress to which the surface course is subjected by traffic.

2. While the quantities c and Φ for any bituminous mixture can be measured by the triaxial test, no method for determining the value of the lateral support L that can be provided by the pavement surrounding the loaded area has been indicated. Unless values for lateral support L can be determined, stability equations (2) and (3), and the stability diagrams based upon these equations (e g. Figs 9, 10 and 11), are of no practical value.

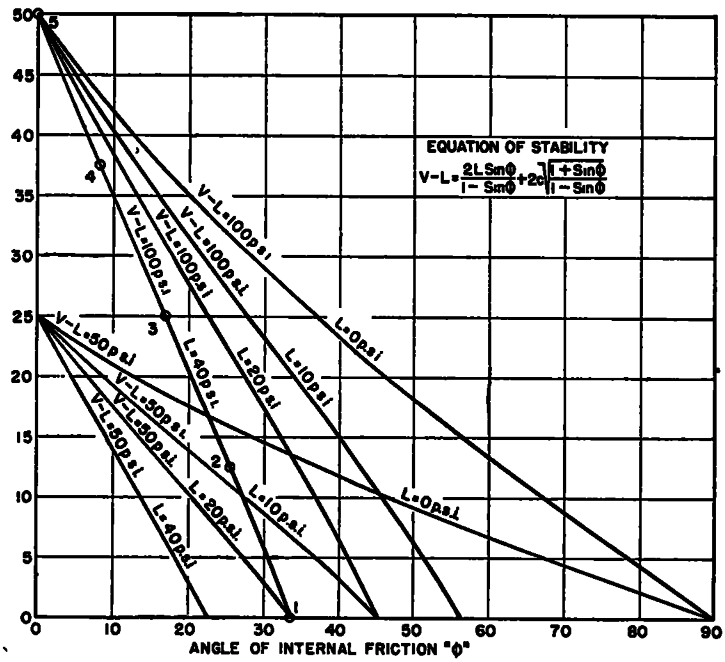

Figure 9. Stability Diagram in Terms of c, ϕ, L and $(V - L)$ for Materials having Positive Values for c and ϕ in Triaxial Compression

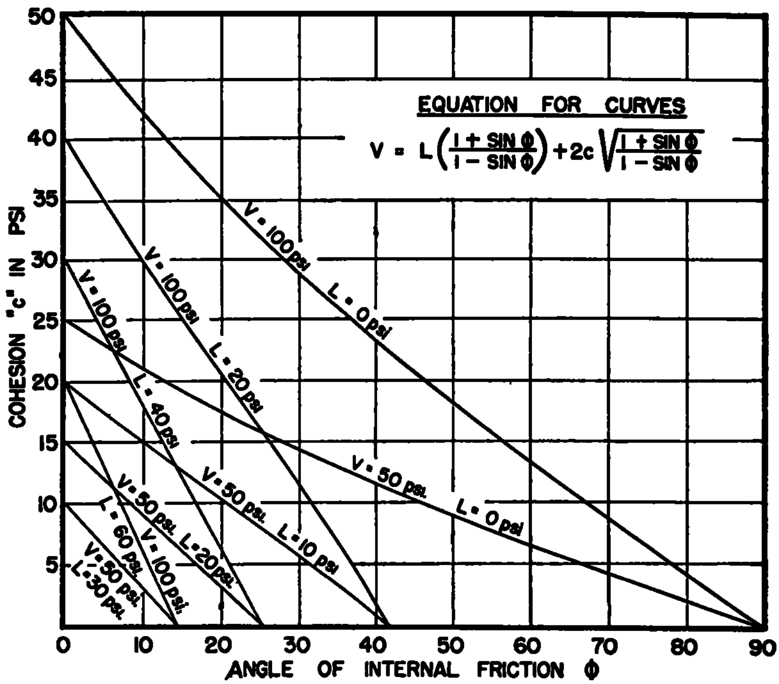

Figure 10. Relationships between c, ϕ, L and V for Materials having Positive Values for c and ϕ in Triaxial Compression

The minimum value of cohesion c required for bituminous mixtures for surface courses may not be particularly high Some years ago, automobile speed record attempts were made on the sandy beach at Daytona Beach, Florida, at a certain time after the tide went out The surface tension of the water retained in the sand over this critical period of time was sufficient to provide the cohesion c (and stability) required for the test run. In addition, it is well known that the moisture films provided or maintained by the application of certain salts, e g. calcium chloride, to the surfaces of stabilized gravel roads prevent the damage to the surfaces of these roads that results when these moisture films are absent. Quantitative values of the cohesion c provided by the moisture films in these cases do not seem to be available, but they are probably not high.

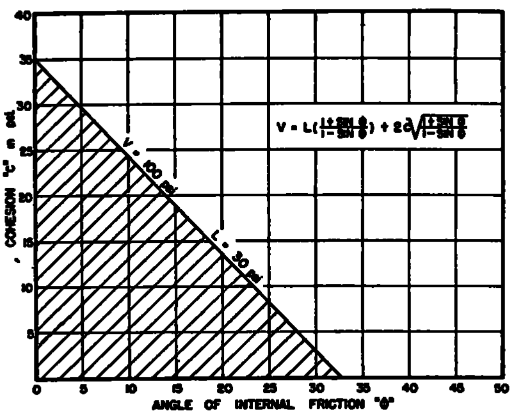

Figure 11. Stability Diagram in Terms of c, ϕ, L and V for Materials having Positive Values of c and ϕ in Triaxial Compression

A method based upon V_t-L_t curves, previously suggested by the writer (*12*) to establish minimum values of cohesion c for bituminous mixtures, may require c values that are higher than necessary. Probably the most reasonable method for obtaining the minimum values of c required for bituminous mixtures, would be to determine them experimentally by means of triaxial tests on samples from bituminous pavements that have performed differently in the field. For example, ravelling may be an indication of insufficient cohesion c in the paving mixture.

A method for determining the maximum value of the lateral support L provided by the portion of the pavement surrounding the loaded area is illustrated in Figure 12 In Figure 12(a) the principal, shear, and normal stresses that are developed in a bituminous pavement under load are indicated when the weight of the pavement material is neglected As the stress caused by the vertical load V develops the shearing resistance s_c on the diagonal plane making an angle of $45 - \frac{\phi}{2}$ with the vertical in element (1), lateral pressure L is exerted in a horizontal direction on element (2) immediately adjacent to the loaded area. The maximum lateral pressure L that can be sustained by element (2) is determined by its shearing resistance s_c acting along the diagonal plane making an angle of $45 - \frac{\phi}{2}$ with the horizontal. Figures 12(b) and (c) illustrate the principal, shear, and normal stresses acting on the isolated elements (1) and (2) respectively, neglecting their weight. Figure 12(d) is a Mohr diagram representing

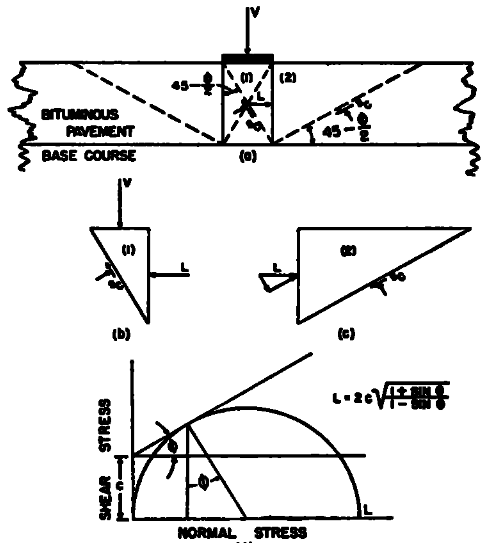

Figure 12. Illustrating that the Lateral Support L provided by the Portion of a Bituminous Pavement surrounding the Loaded Area is given by $L = 2c\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}$

the principal stresses acting on element (2) for a bituminous paving mixture having the values of c and ϕ that result in the Mohr envelope indicated. The values of c and Φ are determined directly from a triaxial test on the mixture. L is the major principal stress acting on element (2), and the minor principal stress is zero, if the weight of the element, and other factors, are neglected. This Mohr circle indicates that the maximum amount of lateral support L that can be developed by the portion of the pavement adjacent to the loaded area is equal to the unconfined compressive strength of the pavement mixture. From the geometry and trigonometry of the Mohr diagram illustrated in Figure 12(d), it is apparent that this value of lateral support L is given by

$$L = 2c\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}. \qquad (4)$$

Substituting this value for lateral support L in equation (3), and simplifying, gives

$$V = \frac{4c}{1 - \sin\phi}\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}. \qquad (5)$$

Equation (5) is the equation of stability for a bituminous paving mixture when it is assumed that the maximum lateral support L provided by the pavement adjacent to the loaded area is equal to the unconfined compressive strength of the paving mixture. This is illustrated in Figure 13.

Figure 14 is the stability diagram that results when equation (5) is plotted in terms of different values of vertical load V and various magnitudes of c and ϕ. If the vertical load V to be carried is 100 psi for example, Figure 14 indicates that only those bituminous mixtures having corresponding values of c and ϕ lying on or to the right of the curve labelled $V = 100$ psi. will provide bituminous pavements with sufficient stability to carry this load.

It should be observed that equation (5) and Figure 14 provide an answer to the two main criticisms of equations (2) and (3), and Figures 9, 10, and 11, as a basis for the design of bituminous mixtures that were previously mentioned The curves in Figure 14, unlike those of Figures 9, 10 and 11, indicate that a certain amount of cohesion c is automatically provided for by the curve for each value of V throughout the diagram In addition, equation (5) implies that the amount of lateral support L provided by the pavement adjacent to the loaded area is equal to the unconfined compressive strength of the material.

While it was developed in connection with Figure 12, that the amount of lateral support L provided by the pavement adjacent to the loaded area is equal to the unconfined compressive strength of the paving mixture, the weight of element (2) was neglected This development was also simplified by neglecting certain other factors that must now be considered, because they indicate that the actual amount of lateral support L provided by the pavement surrounding the loaded area may be

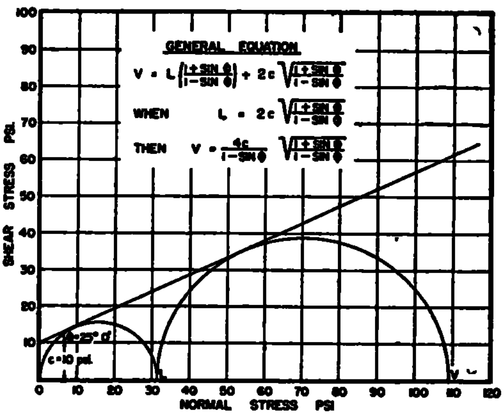

Figure 13. Diagram Illustrating Maximum Vertical Load V that can be carried by a Bituminous Pavement when Lateral Support L is equal to the Unconfined Compressive Strength of the Material

appreciably greater than its unconfined compressive strength.

If the weight of element (2) in Figure 12(a) were taken into account, the minor principal stress acting on this element would not be zero as shown in Figure 12(d), but would have some positive value depending upon the density of the pavement Reference to Figure 12(d) indicates that this would provide a value of lateral support L greater than the unconfined compressive strength, since the Mohr circle corresponding to a minor principal stress greater than zero would be to the right of that shown in Figure 12(d).

Figure 12(a) is based upon a strip loading, and assumes that the stresses applied by a tire have a greater tendency to squeeze a bituminous pavement from under the wheel in a transverse direction than longitudinally towards the front or behind the tire. Figure 15 illustrates the reasonable basis for this assumption, based upon the studies of Teller and Buchanan (*14*) and of Porter (*15*). Towards the front or rear of the contact area of a tire resting on a pavement, the pressure decreases cate that the pressure on the contact area decreases from its full average value to zero over a considerably greater width in a longitudinal than in a transverse direction, and this is supported by Porter's data for a very large airplane tire (*15*). The outer section of the contact area over which the pressure is de-

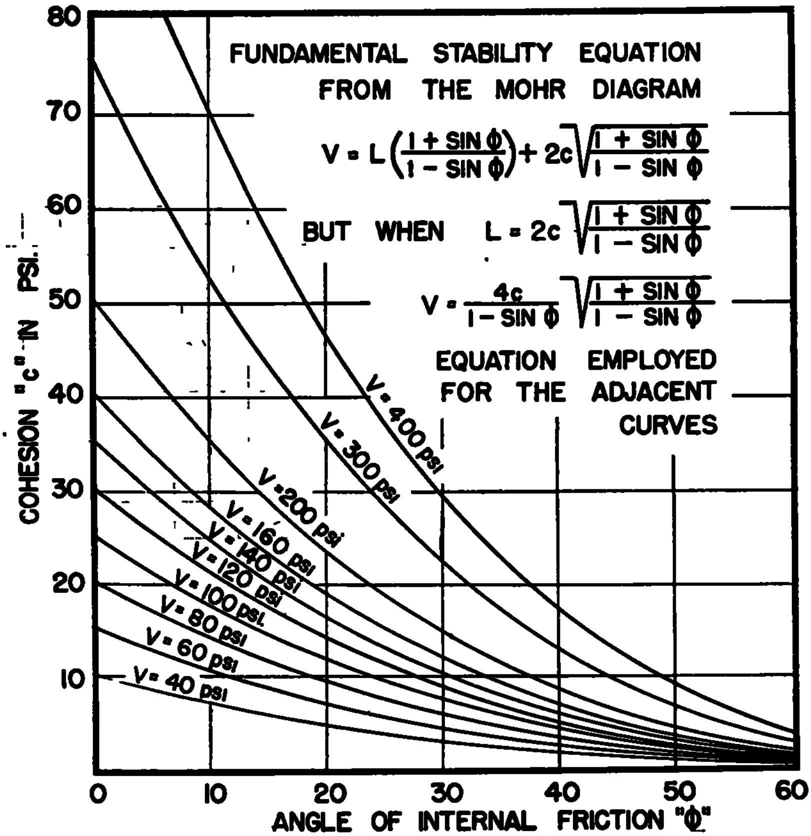

Figure 14. Design Chart for Bituminous Mixtures based on Triaxial Test and Values of Lateral Support $L = 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$

from its full average value to zero over a much longer section of the area of contact, than is the case in the transverse direction. For bus and truck tires equipped with nearly flat treads, the pressure on the contact area probably decreases from its full average value to zero over a very narrow width in a transverse direction Even with the more rounded treads usually employed for airplane tires, the data obtained by Teller and Buchanan (*14*) indicreasing from its full average value to zero, acts as a surcharge on the pavement with respect to the somewhat smaller inner portion of the area of contact over which the full average pressure is exerted. This surcharge effect under a tire is, therefore, acting over a much greater width of pavement in a longitudinal than in a transverse direction Consequently, a bituminous pavement tends to be less stable under the stresses applied by a tire which act in a transverse than in a longitudinal direction, and the former, which corresponds to a strip loading, presents the more critical conditions of stability to be considered in the design of bituminous mixtures, at least for stationary loads. That is, a stationary loaded tire resting on a bituminous pavement has a greater tendency to squeeze out the pavement in a transverse direction than towards the front or rear of the tire

Strip loading assumes a relatively narrow loaded area of indefinite length, whereas the length of a tire contact area is rather short. If the contact area is over-loaded, there is a tendency for a whole wedge of the adjacent pavement, ABCDEF (Fig. 16(a)), to be forced out of the pavement. Consequently, in addition to the resistance due to shear along the diagonal plane ABCD just outside of the loaded area, which provides lateral support L equal to the unconfined compressive strength for a strip load, shearing resistance along the triangular vertical end areas of the wedge, AED and BFC, is also developed. The shearing resistance along the vertical triangular end areas AGD and BHC should also be considered. Depending upon the length of the tire contact area and thickness of pavement, the shearing resistance of these triangular end sections might vary from about 10 percent of the shearing resistance provided by the diagonal plane ABCD (Fig. 16(a)) for a large airplane tire, to about 50 percent for a truck tire. That is, the shearing resistance provided by the vertical end sections AGD and BHC may increase the lateral support L provided by the pavement surrounding the loaded area by from 10 to 50 percent or more of the unconfined compressive strength.

It is apparent from Figure 16(a) that the ratio of the shearing resistance on the vertical triangular end sections AGD and BHC, versus that on the diagonal plane ABCD, would be higher in the direction of the longitudinal rather than the transverse axis of a tire's contact area, since the contact area is narrower in the longitudinal than in the transverse direction. That is, it would provide greater resistance to the squeezing out of a bituminous pavement from under a loaded tire toward the front or rear of the tire, than in the transverse direction.

In an actual pavement the wedge of material under stress just outside the loaded area does not necessarily have the regularly defined shape illustrated by ABCDEF in Figure 16(a), since the contact area of a tire on a pavement is elliptical for airplane tires rather than rectangular as shown, although Paxson (*16*) has demonstrated that the contact areas for heavily loaded truck tires tend to be rectangular. Nevertheless, the important point to be considered in connection with Figure 16(a) is not the actual shape of the wedge of material under stress, but that shearing resistance is developed along the vertical end

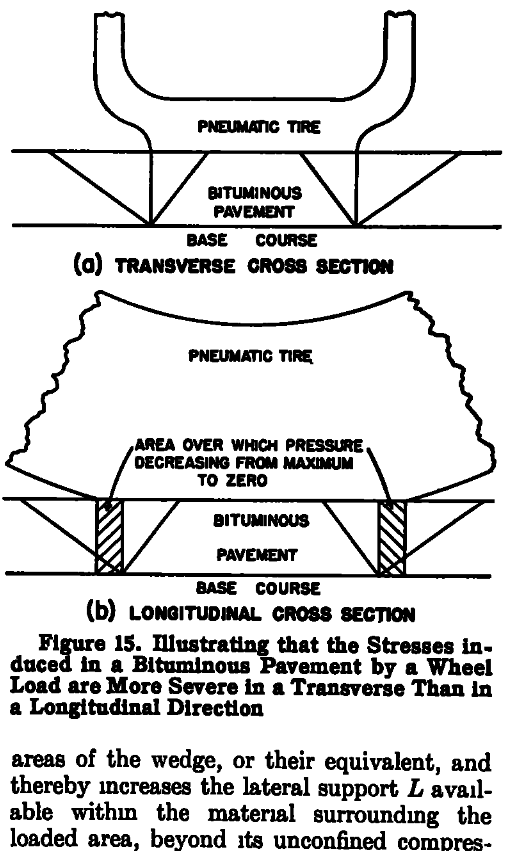

Figure 15. Illustrating that the Stresses Induced in a Bituminous Pavement by a Wheel Load are More Severe in a Transverse Than in a Longitudinal Direction

areas of the wedge, or their equivalent, and thereby increases the lateral support L available within the material surrounding the loaded area, beyond its unconfined compressive strength.

It has been known for many years that a load applied to the surface of a granular mass is spread out over a much wider area on any horizontal plane below the loaded area This is demonstrated in Figure 16(b). It is quite apparent that because of the spreading of the load with depth, the diagonal shear plane bc is subjected to a considerably greater normal stress, than would be the case if this spreading out of the load did not occur. House (*12*) has observed this effect in connection with his investigations of the stability of granular materials. Because of this greater normal stress the shearing resistance along the diagonal shear plane bc is larger than would be the case for an unconfined compression test. Consequently, because of the spreading of the inner portion of the contact area subjected to the full average pressure This surcharge provides a greater normal stress on the diagonal plane bc of Figure 16(b) than would occur with an unconfined compression test. Because of this surcharge factor, the lateral support *L* provided by the pavement surrounding the loaded area is greater than its unconfined compressive strength.

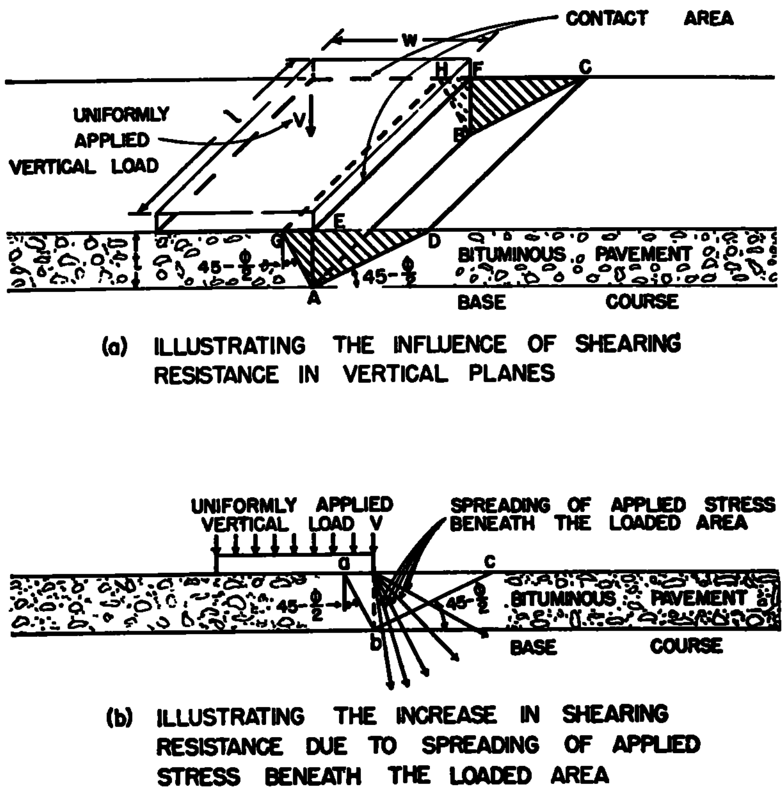

Figure 16. Illustrating Factors that tend to increase the Lateral Support *L* above the Unconfined Compressive Strength of the Pavement

applied load with depth, the lateral support *L* provided by the material surrounding the loaded area is greater than its unconfined compressive strength.

As previously pointed out in connection with Figure 15, there is a narrow band just within the edge of the contact area, over which the pressure decreases from its full average value to zero. The lower pressure on this narrow band acts as a surcharge on the pavement with respect to the somewhat smaller

To summarize, therefore, the lateral support *L* provided by the pavement adjacent to the loaded area is greater than the unconfined compressive strength of the pavement because:

1. The weight of the pavement material increases the normal stress on the plane of failure

2 The area of the vertical and diagonal planes over which shearing resistance is developed is greater than would be the case if this resistance were limited to the unconfined compressive strength for strip loading.

3. The outward spreading of load beneath the loaded area provides a normal stress on the plane of failure which is greater than that which would occur for unconfined compression.

4 The surcharge effect on the outer portion of the contact area over which the pressure is decreasing from its average value to zero, increases the normal stress on the plane of failure.

These additional sources of lateral support L provided by the pavement adjacent to the loaded area can be taken into account by multiplying the unconfined compressive strength by a factor K. That is,

$$L = K\left(2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}\right) \qquad (6)$$

When the value of the lateral support L given by equation (6) is substituted in equation (3), we obtain on simplification

$$V = 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}} \times \left(\frac{K(1+\sin\phi)+(1-\sin\phi)}{1-\sin\phi}\right) \qquad (7)$$

Figure 17 illustrates equation (7) in graphical form, and demonstrates the influence of different values of the factor K varying from 0 to 10 on the design of bituminous mixtures. When $K = 1$, equation (7) reduces to equation (5), so that the stability curve for $V = 100$ psi (when $K = 1$ in Fig. 17) is identical with the curve for $V = 100$ psi. in Figure 14

Stability curves in Figure 17 for $V = 100$ psi when $K = 0$ and when $K = \frac{1}{2}$, apply to the condition where the wheel load is to be applied at or near the unsupported edge of a bituminous pavement. For a wheel load immediately at the unsupported edge of a pavement, the value of K to be employed would approach zero, since the amount of lateral support L provided under these conditions would also approach zero

It will be observed in Figure 17 that each stability curve automatically specifies a minimum value of cohesion c for all values of internal friction ϕ less than 90 deg Whether or not the values of cohesion c indicated by these curves would be adequate in all cases can only be determined from observations of

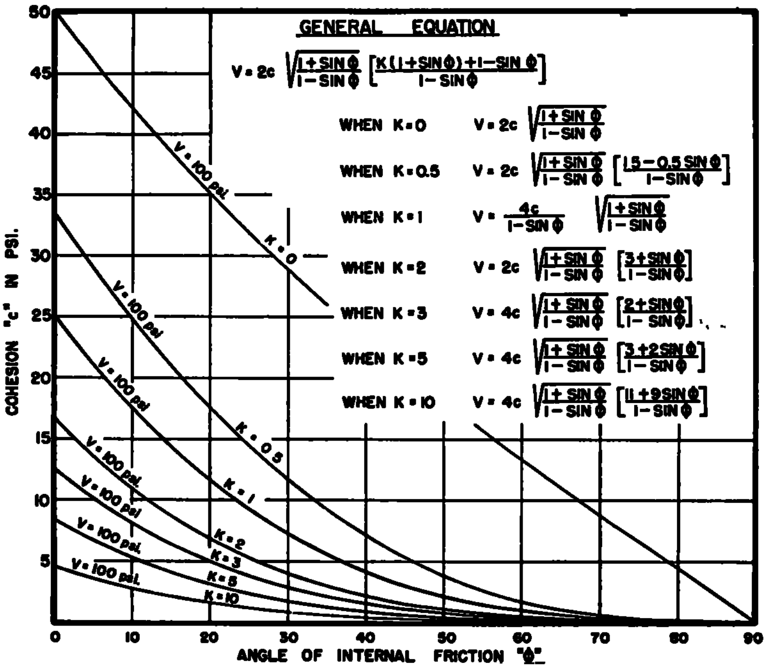

Figure 17. Diagram Illustrating the Influence of Different Values of K on the Design of Bituminous Mixtures

the field performance of bituminous mixtures with known values of c and ϕ. Practical experience might indicate the necessity for arbitrarily specifying some minimum value of cohesion c for all bituminous mixtures, for example 5 psi. It seems more likely, however, that any arbitrarily specified value of cohesion c should vary with angle ϕ and with the applied vertical load V, in some manner to be determined experimentally.

The sources of lateral support L that are taken into account in equation (6) have reference to pavement stability in a transverse direction under a loaded area. For reasons previously outlined, pavement stability is considered to be more critical in a transverse than in a longitudinal direction under a stationary tire load. The sources of lateral support L in a longitudinal direction, that are provided by the pavement adjacent to the loaded area can be included by multiplying the unconfined compressive strength by the factor J, giving,

$$L = J\left(\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}\right) \qquad (8)$$

Since pavement stability appears to be more critical in a transverse than in a longitudinal direction, it seems reasonable to assume that the factor J in equation (8) is larger than K in equation (6). Thus, the latter equation represents a smaller and, therefore, more critical degree of lateral support L than the former.

VISCOUS RESISTANCE

Cohesion c and angle of internal friction ϕ are the two fundamental properties of bituminous mixtures that must be considered when designing their strength or stability on a pounds per square inch basis. These two properties should, therefore, be very carefully measured by a triaxial test on every proposed bituminous paving mixture.

The bituminous binder employed for bituminous mixtures is a very viscous material. While a bituminous paving mixture is deformed, this highly viscous binder provides the mixture with a "viscous resistance" that is proportional to the rate of deformation. The magnitude of the viscous resistance measured for a bituminous mixture depends, therefore, upon the rate of strain or rate of loading employed when making the triaxial test. The viscous resistance of bituminous mixtures becomes important particularly when considering the stability of bituminous pavements subjected to rapidly moving loads.

Consequently, the strength or stability of bituminous mixtures depends upon the magnitude of the three fundamental sources of stability that they possess: cohesion; angle of internal friction; and viscous resistance.

How is this viscous resistance factor to be evaluated in quantitative terms when measuring the stability of bituminous mixtures? A satisfactory approach to this problem seems to be indicated by the work of Nijboer *(10)* and is briefly outlined here.

For measuring the stability of bituminous mixtures, Nijboer employs the "cell" triaxial test (Fig. 18(a)) devised by Buisman at Delft, Holland, some years ago. An outstanding advantage of the cell triaxial test is that a complete Mohr diagram can be obtained with a single test specimen, instead of the three or four specimens required by the more standard type of triaxial equipment employed in North America. The cell triaxial test is described and its use illustrated by reference to Figure 18(a) and (b).

With the cell triaxial test, the prepared specimen is placed in the rubber membrane in the apparatus shown in Figure 18(a). Water or other suitable liquid is pumped into the annular space between the specimen and the outer cylindrical wall. When this space is filled, the valve is closed so that no liquid can escape. Specimen and liquid should be maintained at the desired testing temperature, considered to be critical for the region in which the pavement is to be built.

A constant vertical load V_1 is applied to the test specimen. The specimen deforms rapidly at first under this constant load and builds up lateral pressure L in the surrounding liquid which cannot escape. Finally the rate of deformation slows to zero (no vertical movement under the constant vertical load V_1), and the lateral pressure reaches its maximum value L_1, which is read off the pressure gauge. Since the rate of deformation of the specimen is zero, no viscous resistance is developed within the specimen. The values for V_1 and L_1 can be marked on the abscissa of the Mohr diagram (Fig. 18(b)) and the resulting Mohr circle drawn.

With the constant vertical load V_1 still being maintained, the needle valve is opened slightly to drain off liquid from the space surrounding the specimen at a rate that causes it to be deformed in a vertical direction at some desired rate of strain This constant rate of vertical deformation of the specimen brings its viscous resistance into play, and the constant vertical load V_1 is now resisted partly by the lateral pressure L of the surrounding fluid and partly by the developed viscous resistance of the material. At this constant vertical load V_1 and constant rate of vertical deformation, the lateral pressure drops below its previous value L_1. When the new lateral pressure L has become constant under these conditions, its value L_2 is read from the pressure gauge. The value for L_2 is marked on the Mohr diagram (Fig. 18(b)) and the Mohr circle for V_1 and L_2 is described as shown.

The needle valve is closed and the same procedure is repeated for a new constant vertical load V_2. This gives lateral support values of L_3 for equilibrium conditions (when vertical rate of deformation becomes zero), and L_4 when the material is deformed at the same rate of strain employed for V_1 and L_2 The Mohr circles for V_2 and L_3 and for V_2 and L_4 are drawn on the Mohr diagram

This procedure should be repeated for still higher values of constant vertical load V to obtain additional Mohr circles in order that the Mohr envelopes may be established with greater accuracy. Several different rates of vertical deformation may also be employed to investigate the behavior of any proposed bituminous paving mixture over a wide range of conditions.

Mohr envelopes are drawn tangent to the Mohr circles V_1L_1 and V_2L_3, and to V_1L_2 and V_2L_4 (Fig. 18(b)). It will be observed that the angle of internal friction ϕ is the same for both Mohr envelopes. It has been shown this way in Figure 18(b) because Nijboer (*10*) reports a theoretical study (supported by some test data) which indicates that the angle of internal friction ϕ should be independent of the rate of deformation of the bituminous mixture as long as the air voids are not below the critical minimum, which is usually 2 to 3 percent. With respect to cohesion c on the other hand, it will be noted that cohesion c_2 for the Mohr envelope for Mohr circles V_1L_2 and V_2L_4 is much greater than cohesion c_1 for the Mohr envelope for Mohr circles V_1L_1 and V_2L_3. Consequently, Figure 18(b) indicates that the viscous resistance developed by deforming bituminous paving mixtures at any constant rate of strain is represented by an increase in the value of cohesion c obtained for the mixture.

Stability equations (5) and (7) show that cohesion c is a multiplier for the balance of the expression on the right hand side of each equation. Therefore, any triaxial testing procedure that specifies an unduly high rate of strain will provide enhanced values of cohesion c, which when substituted in stability equations (5) and (7) may indicate stability

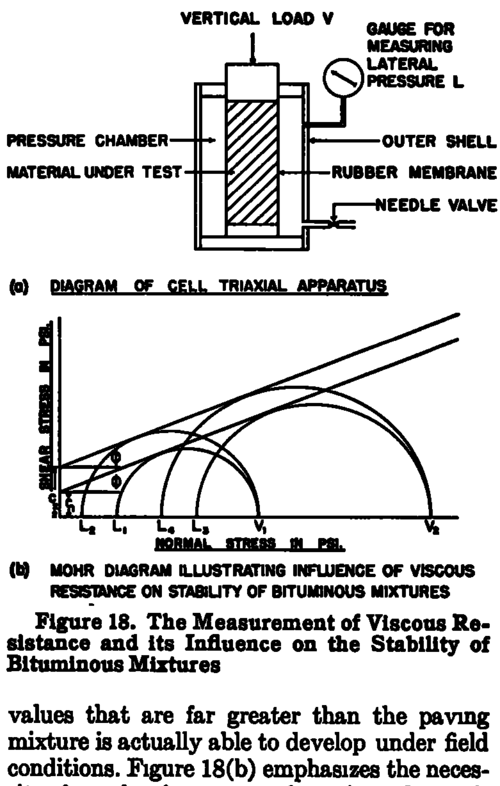

Figure 18. The Measurement of Viscous Resistance and its Influence on the Stability of Bituminous Mixtures

values that are far greater than the paving mixture is actually able to develop under field conditions. Figure 18(b) emphasizes the necessity for adopting rates of strain, when triaxially testing bituminous mixtures in the laboratory, that correspond to the rates of strain to which they will be exposed under traffic in the field.

We are indebted to Mr. John Walter, Assistant Highway Engineer, Department of Highways of Ontario, for the triaxial data that form the basis for Figure 19 The figure illustrates the influence on the Mohr envelope of increasing the rate of strain employed for a

triaxial test on a bituminous concrete paving mixture from 0 05 to 0 4 in per min , that is, one platen of the testing machine moved at rates of 0 05 and 0 4 in. per min. with respect to the other. The specimens used were identical in every respect and were 8 in high by 4 in. in diameter The open triaxial apparatus illustrated in Figure 2 was employed.

The Mohr envelopes of Figure 19 demonstrate that the value of cohesion c obtained for this bituminous mixture was practically doubled, from 19.75 to 38.75 psi., when the rate of strain was increased from 0 05 to 0.4 in. per min. The angle of internal friction ϕ is shown to have decreased several degrees for the higher rate of strain This is not in harmony with Nijboer's conclusions from his theoretical study that the magnitude of the angle of internal friction Φ is independent of the rate of strain (Fig 18(b)) More laboratory work is required to determine whether Nijboer's theoretical deductions hold in all cases, or whether the difference in the angle of internal friction Φ indicated in Figure 19 is either of normal or infrequent occurrence, or if it is due to experimental error

In connection with Figure 19, it is of more than usual interest to observe that the rate of strain employed for the Hubbard-Field stability test for an asphaltic concrete briquette 3 in. in thickness is 2.4 in per min. (*17*). For the specimens 8-in. high on which Figure 19 is based, the rate of strain corresponding to the Hubbard-Field procedure would be 6 4 in. per min The Marshall test employs a rate of strain of 2 in. per min. for a specimen 4 in. high (*18*) This corresponds to a rate of strain of 4 in. per min. for a specimen having a height of 8 in. It is apparent from Figure 19 that rates of strain of 6 4 and 4 in. per min., equivalent to Hubbard-Field and Marshall test procedures, respectively, might give Mohr envelopes that would be well above the top of the diagram The corresponding values of cohesion c might also be above the highest ordinate value shown in Figure 19. Consequently, there seems to be considerable justification for the criticism frequently made of both Hubbard-Field and Marshall tests, that the stability values they provide are influenced very largely by the cohesion c of bituminous mixtures

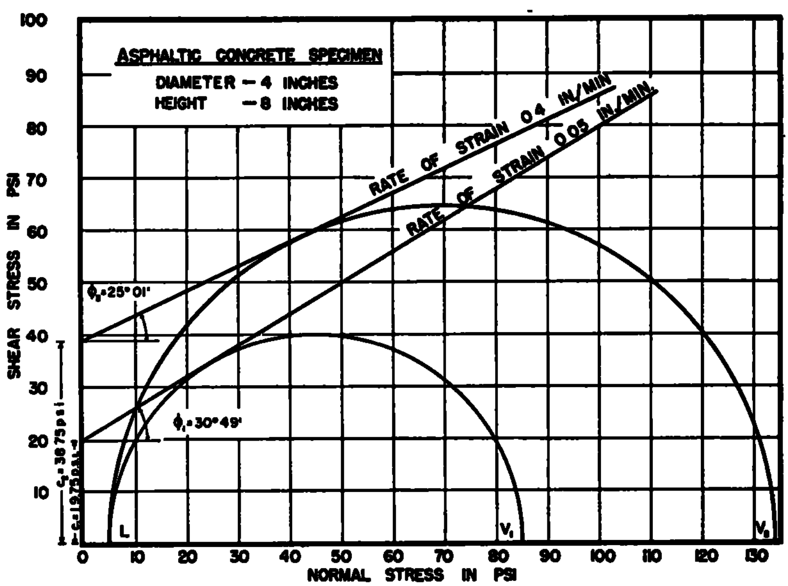

Figure 19. Influence of Rate of Strain on the Stability of Bituminous Mixtures

For the Hveem stabilometer (*3*) the rate of strain employed is 0 05 in. per min for a specimen about $2\frac{1}{16}$ in high. This corresponds to a rate of strain of slightly less than 0 2 in. per min. for a specimen 8 in. tall. For this rate of strain the resulting Mohr envelope would be between those shown in Figure 19

Smith (*4*) has recommended that bituminous mixtures be tested at zero rate of strain This would give a Mohr envelope somewhat below that for the strain rate of 0 05 in. per min illustrated in Figure 19.

The above examples indicate that in terms of test specimens 8 in high, stability tests in common use at the present time specify rates of strain varying from 0 to 6.4 in per min. It is apparent from Figures 18(b) and 19 that little fundamental correlation between these various test methods can be expected until rates of strain are standardized The rate of strain selected for laboratory triaxial tests should clearly approximate the rate of strain to which bituminous paving mixtures are subjected by traffic in the field.

For a truck wheel carrying a load of 5,000 lb. and a tire pressure of 70 psi, any given point on the pavement over which this tire passes is subjected to a load of 70 psi for about 0.6 sec. if the truck is travelling at 1 mph., and for about 0 01 sec if it is travelling at 60 mph. Even with a large airplane tire inflated to 100 psi. and carrying a load of 100,000 lb, a given point on a pavement over which it passes is subjected to this pressure for only about 0 03 sec when travelling at 100 mph., and for about 2 sec. when travelling at 1 mph.

It is clear, therefore, that insofar as moving vehicles are concerned, bituminous pavements are subjected to loads of very short duration, and the viscous resistance developed by the bituminous mixture must be quite high. This would be equivalent to conducting a laboratory stability test at a high rate of strain. It might seem, therefore, that a reasonably high rate of strain would be justified for the stability testing of bituminous mixtures to be employed where moving traffic is expected, such as airport runways, and for highways, apart from bus stops and traffic lights However, pavements on airport runways, on highways, and on city streets, are subject to braking and acceleration stresses The influence of these on pavement design is considered in a later section of this paper.

Pavements for stationary or extremely slow moving traffic should be designed on the basis of laboratory stability tests performed at a very low rate of strain Nijboer (*10*) has observed that a pneumatic tire resting on a bituminous pavement 2 in thick that is giving satisfactory service settled about 1 mm into the pavement in 30 min He, therefore, recommends that the rate of strain employed for triaxial tests should be about 0.005 in. per min. for specimens 8 in. high. This rate of strain would result in a Mohr envelope somewhat below the lower Mohr rupture line in Figure 19. The rate of strain recommended by Nijboer for pavement design for stationary loads may not be unreasonable. Nevertheless, it is clear that the rate of strain to be employed for laboratory stability tests requires further careful consideration by everyone interested in this topic.

Nijboer (*10*) indicates that the limits of accuracy when testing successive samples of a given paving mixture with the cell triaxial apparatus, are 30 min for the angle of internal friction ϕ, and 10 percent, or a minimum of 1 4 psi whichever is greater, for cohesion c.

INFLUENCE OF FRICTIONAL RESISTANCE BETWEEN PAVEMENT AND TIRE AND BETWEEN PAVEMENT AND BASE

In a previous section, the influence on pavement stability of the lateral support L provided by the pavement adjacent to the loaded area was considered. However, there are frequent examples where even at the completely exposed and unsupported edge of a bituminous pavement, no indications of instability have developed after years of traffic. These unsupported pavement edges are stable under traffic, either because of the high compressive strength of the paving mixture, or because bituminous pavements can develop additional resistance to lateral flow, quite apart from the lateral support normally provided by adjacent pavement material, or both. Figure 20 indicates that there is a further source of resistance to the lateral movement of a pavement under a loaded area that must be considered.

Figure 20(a) illustrates the resistances developed when a horizontal force L is applied to an isolated section of bituminous pavement held between two rough flat surfaces carrying a vertical load. It is apparent that the horizontal pressure L applied as shown, will develop frictional resistance s between the pavement and each of the two rough surfaces. That is, frictional resistance can be developed between the pavement and the two rough surfaces equivalent to a horizontal pressure L.

In Figure 20(b) the section of bituminous pavement is subjected to sufficient vertical load V to cause it to flow laterally This is equivalent to the movement of an overloaded bituminous pavement beneath a tire. Figure 20(b) demonstrates that as the paving mixture is being squeezed out, its lateral movement is opposed by the frictional resistance s developed between pavement and tire and between pavement and base. It is apparent from both Figures 20(a) and (b) that this frictional resistance between pavement and tire and pavement and base is equivalent to a lateral support L_R.

If this frictional resistance is to be utilized for the design of bituminous mixtures, it must be evaluated quantitatively and taken into account in equations of design and when constructing charts of design curves. Figure 21 illustrates a method for evaluating this frictional resistance in terms of an equivalent lateral support L_R.

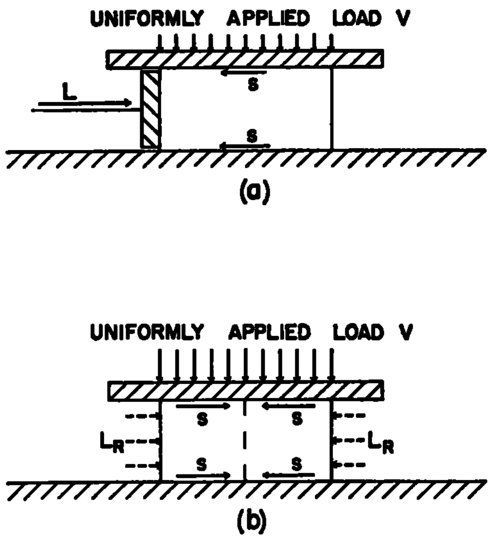

Figure 20. Diagram Illustrating that Friction between Tire and Pavement and between Pavement and Base is equivalent to additional Lateral Support for the Section of Pavement under a Loaded Area

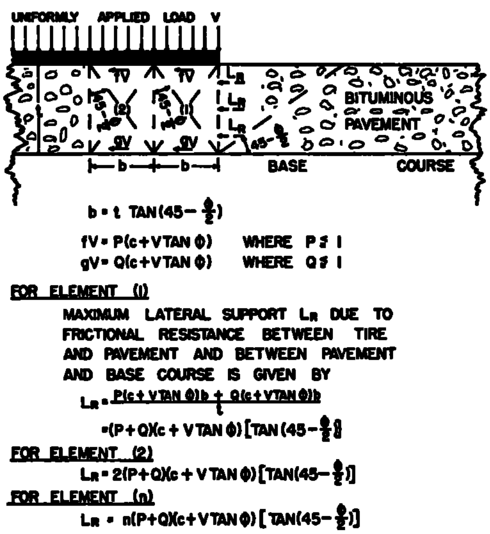

Figure 21. Diagram Illustrating the Magnitude of the Lateral Support L_R Equivalent to the Frictional Resistance Developed between Tire and Pavement and between Pavement and Base, under the Loaded Area

Krynine (*19*) refers to the work of Jurgenson (*20*) which shows that when a material is squeezed between two rough parallel plates, the shearing stress developed in the material is at a minimum on the plane parallel to and midway between the two plates The maximum shearing stress occurs at the boundaries between the plates and the material. In the case of a bituminous pavement squeezed between a tire and the base course, it seems reasonable to assume, therefore, that the maximum shearing stress is developed at the interfaces between pavement and tire and between pavement and base.

It should be clear that the maximum frictional resistance that can be developed at these two interfaces, cannot exceed the shearing resistance of the pavement itself. That is, the maximum frictional resistance that can be developed between the pavement and tire is the lesser of either the coefficient of friction f between pavement and tire multiplied by the normal pressure V (that is fV) or the shearing resistance of the bituminous mixture given by the Coulomb equation, $s = c + V \tan \phi$, where V is the normal pressure. Similarly, the maximum frictional resistance that can be mobilized between the pavement and base is the lesser of either the coefficient of friction g between pavement and base multiplied by the normal pressure V (that is gV) or the shearing resistance of the bituminous mixture given by the Coulomb equation, $s = c + V \tan \phi$.

The limitation on the maximum value of fV that can be developed, can be expressed by letting

$$\frac{fV}{c + V \tan \phi} = P \text{ Where } P \leq 1 \qquad (9)$$

from which

$$fV = P(c + V \tan \phi) \qquad (9a)$$

Similarly, the limitation on gV can be expressed by letting

$$\frac{gV}{c + V \tan \phi} = Q \quad \text{Where } Q \leq 1 \qquad (10)$$

from which

$$gV = Q(c + V \tan \phi) \qquad (10a)$$

Equations (9) and (10) state that the coefficient of friction between pavement and tire f and between pavement and base g multiplied by the normal stress V, cannot exceed the shearing resistance of the bituminous paving mixture, although the reverse could occur, both of which, of course, are true in actual practice The terms P and Q express the values of the ratios of equations (9) and (10). It is apparent that the highest value either P or Q can have individually is unity and the lowest value is zero. Therefore, the maximum value for $P + Q = 2$, and the minimum value for $P + Q = 0$.

Values of the coefficient of friction f between tire and pavement have been measured by Moyer (*21*) and by Giles and Lee (*22*). They report values of f up to 1.0 for stationary or slowly moving vehicles, although 0.8 is a more normal top value. Moyer's data indicate that the value of the coefficient of friction f drops appreciably as the speed of the vehicle increases. No data are available concerning the value of g, the coefficient of friction between pavement and base. Both f and g could be evaluated either in the laboratory or on actual projects in the field.

In Figure 21 the pavement under the loaded area has been divided into several elements numbered inward from the edge It is assumed that the pavement material under the loaded area tends to fail along planes making an angle of $45 - \frac{\phi}{2}$ with the vertical. If the thickness of the pavement is t in., and if the width of each element under the loaded area is b in , it is apparent that,

$$b = t \tan\left(45 - \frac{\phi}{2}\right) \qquad (11)$$

from which it follows that the ratio

$$\frac{b}{t} = \tan\left(45 - \frac{\phi}{2}\right) \qquad (12)$$

The other equations for determining the value of the lateral support L_R, that is equivalent to the frictional resistance between pavement and tire, and between pavement and base, are listed in Figure 21, for successive elements of the pavement numbered inward from the edge of the loaded area. The general equation for L_R for the nth element from the edge is;

$$L_R = n(P + Q)(c + V \tan \phi) \times \left[\tan\left(45 - \frac{\phi}{2}\right)\right] \qquad (13)$$

Consequently, the total lateral support L that can be mobilized for the stability of bituminous mixtures is given by the sum of the lateral support L_S provided by the pavement adjacent to the loaded area, plus the lateral support L_R equivalent to the frictional resistance between pavement and tire and between pavement and base, or

$$L = L_S + L_R \qquad (14)$$

where L_S is given by equation (6) or (8) and L_R by equation (13).

When the expression for the total lateral support L given by equation (14) is substituted in equation (3), we have,

$$V = 2cK\sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \cdot \frac{1 + \sin \phi}{1 - \sin \phi} + 2c\sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} + n(P + Q)(c + V \tan \phi)\left(\tan\left(45 - \frac{\phi}{2}\right)\right)\frac{1 + \sin \phi}{1 - \sin \phi} \qquad (14a)$$

which on simplification becomes,

$$V = c\sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}}\left[\frac{n(P + Q) + 2(1 - \sin \phi) + 2K(1 + \sin \phi)}{1 - (1 + n(P + Q)) \sin \phi}\right] \qquad (15)$$

It is instructive to determine the stability of successive elements of a bituminous pavement inward from the edge of the loaded area. This is necessary if the least stable element under the loaded area is to be determined

Equation (15) cannot be used for this purpose, because it applies only to the determination of the corresponding values of c and ϕ required to support some specified unit load V uniformly applied to the contact area, when values for n, K, and $P + Q$ are also given (e g Fig. 28). For investigating the change in stability across the loaded area, on the other hand, a bituminous mixture with given values of c and ϕ must be considered, and the pavement stability developed at various points on the contact area may be quite different from the uniformly applied unit load. To determine the stability of successive elements of a bituminous pavement inward from the edge of the loaded area, therefore, it is necessary to rewrite equation (15) in somewhat different form, when it becomes,

$$V = 2cK\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}\cdot\frac{1+\sin\phi}{1-\sin\phi} + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}} + n(P+Q)\,(c + V'\tan\phi) \cdot \left(\tan\left(45-\frac{\phi}{2}\right)\right)\frac{1+\sin\phi}{1-\sin\phi} \quad (15a)$$

where

V = the stability developed by the bituminous pavement at any point on the contact area

V' = the unit vertical load uniformly applied to the contact area

and the other symbols have the significance previously defined for them.

It should be noted that in equation (15), which is employed for pavement design, $V = V'$, whereas this is not generally true for equation (15a).

For specified values for c, ϕ, K, $P + Q$, and V', it should be apparent that the value of V in equation (15a) varies directly and linearly with the value of n, where n indicates the distance measured in unit elements (Fig. 21) from the edge to the point on the contact area at which the stability value V is required. This is illustrated by the straight line stability curves in Figures 22, 23, and 24, for a large airplane tire, and Figure 25 for a truck tire, all of which were determined by means of equation (15a)

The values of c = 7 21 psi and ϕ = 30 deg. enable the bituminous pavement in Figure 22 to just support an applied vertical load of

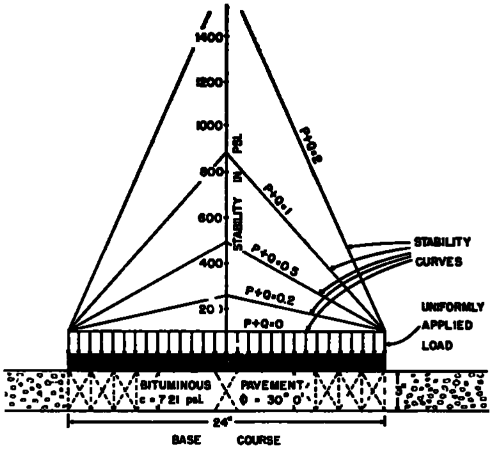

Figure 22. Relationships between applied Load and Stability of Bituminous Pavements at varying distances from Edge under the Loaded Area and for Different Degrees of Frictional Resistance developed between Pavement and Tire and between Pavement and Base (Pavement Stability equal to Applied Load for Edge Conditions) Airplane Tire

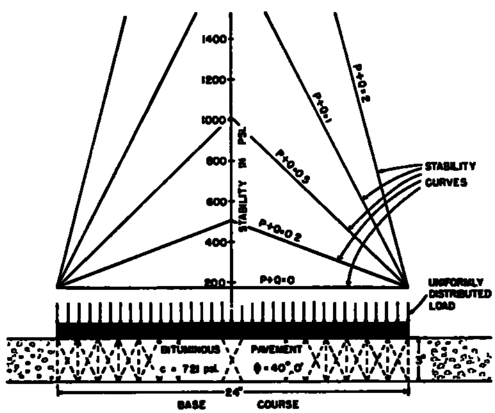

Figure 23. Relationships between Applied Load and Stability of Bituminous Pavements at varying Distances from Edge under the Loaded Area and for Different Degrees of Frictional Resistance between Pavement and Tire and between Pavement and Base (Pavement Stability greater than Applied Load for Edge Conditions) Airplane Tire

100 psi. on an airplane tire contact area 24 in. wide, if the lateral support L is equal to the unconfined compressive strength of the pavement ($K = 1$ in equation (6)). Consequently, if no frictional resistance is developed between pavement and tire and between pavement and base ($P + Q = 0$) this pavement will just carry the applied vertical load V of 100 psi. However, if this frictional resistance is only great enough to make $P + Q = 0.2$ (equations (9) and (10)), it is apparent from the stability curve labelled $P + Q = 0.2$ in Figure 22 that the stability of the pavement under the loaded area increases quite rapidly with distance inward from the edge. This improvement in

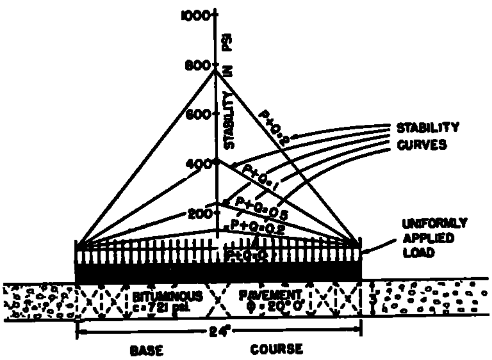

Figure 24. Relationships between applied Load and Stability of Bituminous Pavements at Varying Distances from Edge under the Loaded Area for Different Degrees of Frictional Resistance developed between Pavement and Tire and between Pavement and Base (Pavement Stability less than Applied Load for Edge Conditions) Airplane Tire

stability with increasing distance inward from the edge is still more rapid for values of $P + Q = 0.5$, $P + Q = 1.0$ and $P + Q = 2.0$, as illustrated by the corresponding stability curves in Figure 22.

Figure 23 is similar to Figure 22, with the exception that the pavement stability even at the edge, about 173 psi., is considerably greater than the vertical load $V = 100$ psi. to be carried. This represents a condition of overdesign. The stability curves for all positive values of $P + Q$ are steeper in Figure 23 than in Figure 22.

Figure 24 represents a condition of pavement underdesign for edge conditions, since the pavement stability is only about 63 psi. at the edge, whereas the applied load $V = 100$ psi. The stability curve labelled $P + Q = 0.2$ indicates that even for the amount of frictional resistance between pavement and tire and between pavement and base represented by $P + Q = 0.2$, (equations (9) and (10)), only at a very considerable distance inward from the edge does the pavement develop stability equal to the applied load V.

Figure 25 has reference to pavement stability for the much smaller contact area of truck tires. With respect to the relationships

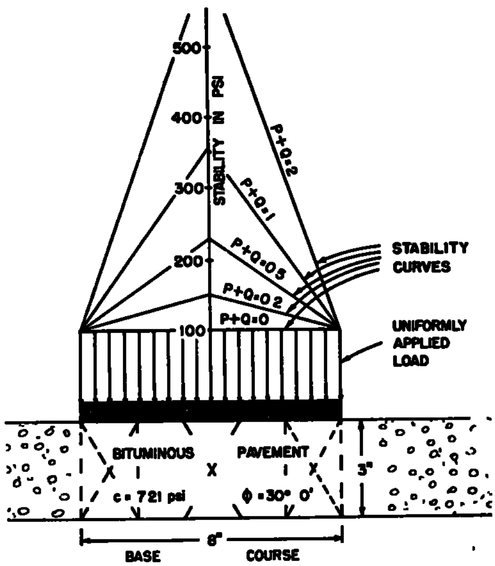

Figure 25. Relationships between Applied Load and Stability of Bituminous Pavements at Varying Distances from Edge under the Loaded Area and for Different Degrees of Frictional Resistance developed between Pavement and Tire and between Pavement and Base (Pavement Stability equal to Applied Load for Edge Conditions) Truck Tire

between the stability curves for values of $P + Q$ varying from 0 to 2, it is quite similar to Figure 22 for the larger contact area of large airplane tires.

The important conclusion to be drawn from Figures 22, 23, 24 and 25 is that for a uniformly applied load the portion of the bituminous pavement just under the edge of the loaded area is the most critical insofar as the stability of bituminous mixtures is concerned. At any distance inward from the edge the bituminous pavement under the loaded area tends to develop increased stability if there is frictional resistance between pavement and tire and between pavement and base, the actual increase in stability depending upon the distance from the edge, and upon the value of the frictional resistance between pavement and tire and pavement and base, represented by $P + Q$.

Depending upon the magnitude of the frictional resistance developed between pavement and tire and between pavement and base, Figures 22, 23, 24, and 25 indicate that the ratio of width of tire contact area to thickness of pavement may to some very considerable degree determine whether a given bituminous mixture provides a stable or an unstable pavement This seems to be quite in keeping with practical experience. Properly constructed surface treatments for example, even when made with sand cover material and in spite of their high binder content, do not tend to squeeze out even under relatively narrow truck tires. The high stability developed a very short distance inward from the edge of the contact area, illustrated in Figures 22, 23, 24, and 25, provides a reasonable explanation for this behavior. Similarly, a bituminous mixture that shows questionable stability when laid as a thick pavement, may develop quite adequate stability when placed in a relatively thin layer. Figures 22, 23, 24 and 25 also indicate that a given bituminous paving mixture might have adequate stability on the paved area of an airport under wide airplane tires, but be quite unstable under the much narrower truck tires inflated to the same tire pressure, if employed for a highway pavement, since the proportion of understressed pavement under the loaded area could be much less for the latter than the former.

Since bituminous paving mixtures apparently develop their lowest stability near the edge of the contact area, the conditions of stability across the first element just within the edge of the loaded area should be examined. These are illustrated in Figure 26. The exact location of this first element is shown in Fig. 26(a). The relationship between b, t and the potential angle of failure, $45 - \phi/2$, for this element is given by equation (11). The symbol n refers to the number of the element of width b under consideration, with the numbering beginning from the edge of the loaded area (Fig. 21) Thus $n = 1$ for the first element just within the edge of the contact area, $n = 2$ for the second element, etc. It is apparent that n can also have fractional values.

Figure 26(b) illustrates the value of L_R distributed uniformly across the vertical face of length t of element (1) ($n = 1$) that is equivalent to the frictional resistance fV acting over the width b between pavement and tire, and gV acting over width b between pavement and base. For the fraction of a unit element represented by $n = \frac{1}{2}$, Figure 26 (c) demonstrates that the width of this element is only $b/2$. Consequently, the total frictional

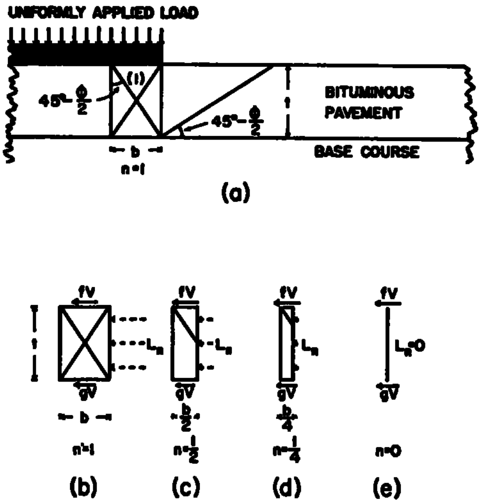

Figure 26. Illustrating, over the First Element, the Influence of Distance from the Edge on the Magnitude of the Lateral Support L_R due to Frictional Resistance between Pavement and Tire and Pavement and Base

resistance between pavement and tire and between pavement and base, and therefore L_R, is only one half of that for the full unit element Figure 26(b). Therefore, when $n = \frac{1}{2}$, the magnitude of L_R is reduced to one half of the value it had for the full unit element, $n = 1$. Similarly, Figure 26(d) shows that for the fraction of a unit element represented by $n = \frac{1}{4}$, L_R has only one quarter of its value for the full unit element. Finally, when $n = 0$, which represents the vertical plane through the edge of the contact area, $b = 0$, and therefore, $L_R = 0$. Consequently, over the first element within the edge of the loaded area, the value of L_R varies from zero at the edge, to a maximum at a distance b in from the edge, where b is the width of the unit element, $n = 1$, as illustrated in Figure 26(a).

Since L_R is zero at the exact edge of the contact area, it might seem reasonable to completely disregard the frictional resistance between pavement and tire and pavement and base as a source of lateral support in the design of bituminous mixtures. This would imply that the only lateral support L available is that provided by the pavement adjacent to the loaded area. A conservative design could be made on this basis, and it would utilize equation (7) as the stability equation to be employed.

Nevertheless, further consideration of the conditions of loading near the edge of the loaded area seem to indicate that some portion of the frictional resistance between pavement and tire and between pavement and base does contribute to pavement stability, and that some value for L_R is usually justified for design.

The pressure contour lines over the tire contact area indicated by the investigations of Teller and Buchanan (*14*) and of Porter (*15*) demonstrate that there is an area of some width just inside the edge of the contact area over which the pressure drops from its full average value to zero. This is particularly true of the rounded tires generally employed for aircraft and may be somewhat less so for truck and bus tires equipped with much flatter treads. In both of the investigations just referred to, the maximum pressure V on the contact area occurred at some distance in from the edge in a transverse direction across the contact area, and is thought to be due to the stiffness of the side walls of the tires. Consequently, for airplane tires in particular, and probably for truck and bus tires with flatter treads as well, the maximum vertical pressure on the pavement is exerted at some distance inward from the edge of the contact area, that is, at some value of n (Fig. 26). Between this point and the edge of the loaded area the contact pressure drops gradually to zero. Consequently, if design should be based upon the maximum pressure V on the contact area, some value for L_R is developed between the point where this maximum pressure occurs and the edge of the loaded area.

The actual distance n, measured in unit elements (Fig. 26) from the edge of the loaded area to the point on the contact area at which the critical vertical pressure V occurs, on which design should be based probably varies from tire to tire, and with the conditions of loading. The determination of the variation in this distance n from the edge, and of the average value of L_R to be employed over this distance, are matters that require some experimental study.

For the design curves based upon equation (15), shown in Figure 27 to illustrate the influence on pavement stability of frictional resistance between pavement and tire and between pavement and base, it has been assumed that $n = 1$, that is, the critical point of loading is at a width of one unit element within the edge of the loaded area (Fig. 26) and that the load is uniformly distributed over the contact area. This leads to a somewhat smaller design load than the maximum that may actually occur on the loaded area, and to a somewhat larger value of L_R than may really be developed. It should be noted, however, that the value of L_R to be utilized for pavement design for any project can be modified as required by adjusting the value of $P + Q$ to be employed. In addition, the value of $K = 1$ has been taken for the design curves shown in Figure 27, and, as pointed out in an earlier section, this value of K seems to be quite conservative.

The curves in Figure 27 illustrate the important influence of frictional resistance between pavement and tire and between pavement and base on the design of bituminous paving mixtures Each curve indicates the minimum values of c and ϕ required to carry a vertical load V of 100 psi., under the conditions assumed, as this frictional resistance is gradually increased from zero ($P + Q = 0$) to its maximum value ($P + Q = 2$). For example, if the cohesion c is 5 psi. in each case, an angle of internal friction ϕ of about 37 deg. is required to carry this vertical load when $P + Q = 0$. ϕ decreases to about 31 deg when $P + Q = 0.25$, to about 26 deg. when $P + Q = 0.5$, to about 19.5 deg. when $P + Q = 1$, and to about 13 deg. when $P + Q = 2$.

Figure 28 contains a series of stability curves based upon equation (15) for values of applied vertical load V varying from 40 to 400 psi., assuming that $n = 1$, $K = 1$, and $P + Q = 0.5$. It should be clear that stability diagrams similar to Figure 28 and based upon equation (15) can be drafted for other values

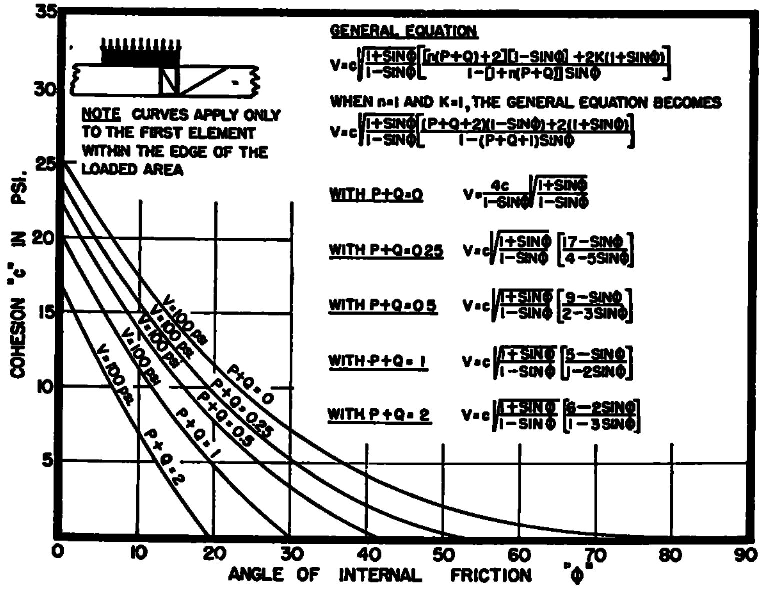

Figure 27. Influence of Frictional Resistance between Pavement and Tire and between Pavement and Base on Design of Bituminous Mixtures to carry a Specific Load

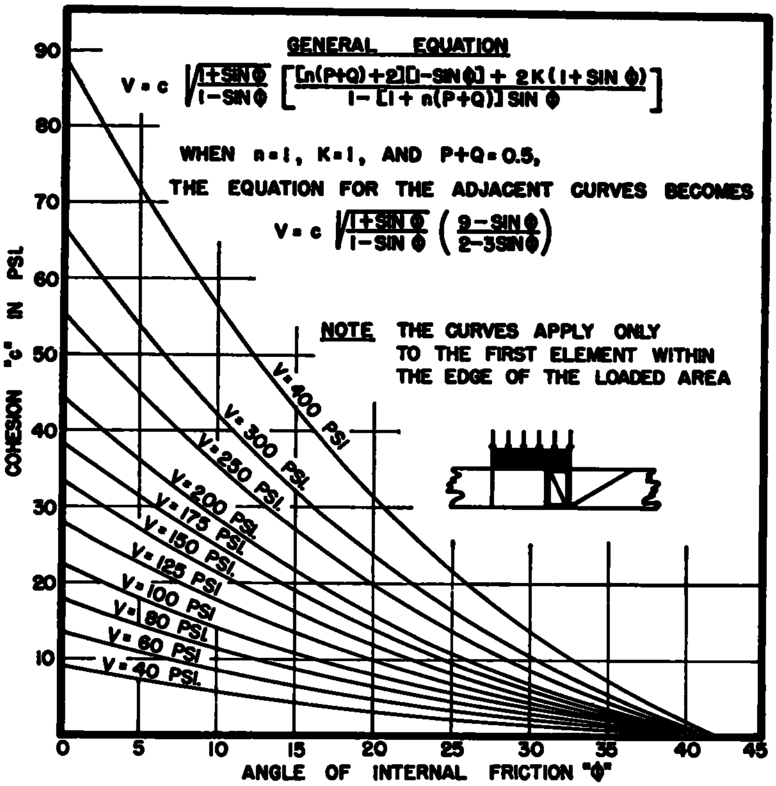

Figure 28. Design Chart for Bituminous Mixtures based on Triaxial Compression Test when $P + Q = 0.5$

of n, K and $P + Q$ Only those bituminous mixtures with combinations of c and ϕ lying on or to the right of any given stability curve will be stable under the vertical load V indicated for that curve.

By comparing Figure 28 with Figure 14 ($K = 1$ for both figures), the importance of frictional resistance between pavement and tire and between pavement and base on the design of bituminous paving mixtures for any applied vertical load V can be observed. In both Figures 27 and 28, it will be noted that the design curves cross the abscissa at values of ϕ within the normal range of those employed for bituminous mixtures, that is cohesion c becomes zero at these values of ϕ. Since bituminous mixtures require some minimum value of cohesion c, it may be necessary to arbitrarily assign a minimum value of c = 5 psi., for example, to all design charts similar to those of Figures 14, 17, 27, 28, etc. However, as previously pointed out, experience may show that any arbitrarily established minimum value of cohesion c should vary with both ϕ and V.

INFLUENCE OF BRAKING STRESSES

In the development outlined so far, a strip loading has been assumed, which implies that upon failure a bituminous mixture will be squeezed from under the tire towards each side of the longitudinal lane followed by the wheel.

In actual service, however, in addition to the tendency to be squeezed from under the wheel at right angles to the direction of travel, bituminous pavements are subjected to braking and acceleration stresses. These latter forces are usually applied in the direction of travel, and they, therefore, attempt to shove the pavement either ahead of (for braking), or behind (for acceleration) the wheel. Since they may often provide the most critical conditions of design for bituminous pavements carrying moving loads, the influence of these braking and acceleration stresses must be considered in a quantitative manner, if possible. From their very nature it is probable that braking stresses are generally more severe than acceleration stresses.

Figure 29 illustrates the forces to be considered for the design of a bituminous pavement capable of resisting braking and acceleration stresses. To simplify the approach to this problem, the contact area between tire and pavement is assumed to be rectangular in shape. Paxson's (*16*) measurements indicate that the tire contact areas of trucks and buses are very nearly rectangular, although for airplane tires they are elliptical If the area of contact is assumed to be rectangular, the element of pavement involved, when considering the influence of braking or acceleration stresses, is the rectangular block of pavement, abcdjihg, immediately beneath the loaded area.

When a braking stress is applied, the two forces tending to shove the paving mixture ahead of the tire are.

1. The horizontal braking stress fV acting on the contact area, bcih

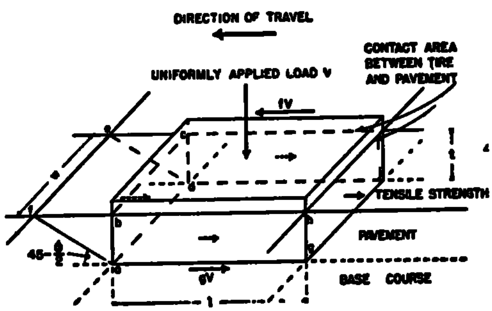

Figure 29. Illustrating the Forces induced in a Bituminous Pavement by Braking Stresses

2. The vertical load V from the pressure of the tire acting on the contact area bcih.

The influence of these two forces can be resolved into equivalent horizontal unit stresses acting toward the left in Figure 29, against the vertical face abcd of the wedge of pavement just ahead of the loaded area The area of abcd is given by wt, where w is the width of the contact area, and t is the thickness of pavement

The unit stress on the rectangular face abcd, corresponding to the braking stress on the contact area, is given by

$$\frac{(fV)(lw)}{wt} = \frac{fVl}{t} \qquad (16)$$

Where f = coefficient of friction between pavement and tire. Its value may be as high as unity, but is usually below 0.8. The value of f tends to decrease as vehicle speed is increased (*21*).

V = average vertical pressure of tire on contact area

l = length of contact area

w = width of contact area
t = thickness of pavement.

However, the maximum braking stress fV cannot exceed the shearing resistance of the pavement under the loaded area, $c + V \tan \phi$. Any tendency for fV to exceed $c + V \tan \phi$ would merely result in shearing of the pavement. This limitation on the maximum value of fV that can be developed, can be expressed by letting

$$\frac{fV}{c + V \tan \phi} = P \qquad \text{Where } P \leqq 1 \tag{9}$$

from which

$$fV = P(c + V \tan \Phi) \tag{9a}$$

Substituting this value for fV in equation (16) gives

$$\frac{fVl}{t} = \frac{P(c + V \tan \phi)\, l}{t} \tag{17}$$

The horizontal unit stress on the rectangular face abcd due to the vertical pressure V of the tire on the contact area is obtained by rearranging equation (3), giving,

$$L = V\frac{1 - \sin \phi}{1 + \sin \phi} - 2c\sqrt{\frac{1 - \sin \phi}{1 + \sin \phi}} \tag{18}$$

where the lateral pressure L is the unit stress required

Consequently, the equivalent horizontal unit stress acting on the rectangular face abcd, due to the combined effect of the vertical load V and the braking force fV and tending to shove the pavement ahead of the loaded area (to the left in Fig 29), is given by the summation of the quantities on the right hand sides of equations (17) and (18), or

$$\frac{P(c + V \tan \phi)\, l}{t} + V\frac{1 - \sin \phi}{1 + \sin \phi} - 2c\sqrt{\frac{1 - \sin \phi}{1 + \sin \phi}} \tag{19}$$

The forces tending to resist the shoving of the pavement ahead of the tire by the vertical load and braking stress are,

1. The frictional resistance gV, between pavement and base, acting over the rectangular surface adjg.
2. The shearing resistance of the paving mixture along both sides, abhg and dcij, of the rectangular block.
3. The resistance to displacement of the wedge of pavement abcdef immediately in front of the loaded area.
4. The tensile strength of the pavement acting on the vertical face ghij at the rear of the rectangular block of pavement under the loaded area.

These four resisting forces can be resolved into equivalent horizontal unit stresses acting toward the right against the front end, abcd, of the rectangular block of pavement under the loaded area.

The horizontal reaction on the rectangular face abcd, equivalent to the frictional resistance between the pavement and base course, is given by

$$\frac{(gV)(lw)}{wt} = \frac{gVl}{t} \tag{20}$$

Where g = coefficient of friction between pavement and base course,

and the other symbols have the significance previously defined for them.

The maximum frictional resistance between pavement and base gV however, cannot exceed the shearing resistance of the pavement under the loaded area, $c + V \tan \phi$. Any tendency for gV to exceed $c + V \tan \phi$ would merely result in shearing within the pavement itself. This limitation on the maximum value of gV that can be developed, can be expressed by letting

$$\frac{gV}{c + V \tan \phi} = Q \qquad \text{Where } Q \leqq 1 \tag{10}$$

from which

$$gV = Q(c + V \tan \phi) \tag{10a}$$

Substituting this value for gV in equation (20) gives

$$\frac{gVl}{t} = \frac{Q(c + V \tan \phi)\, l}{t} \tag{21}$$

The shearing resistance of the paving mixture along the two vertical sides, abhg and dcij, can be obtained from the Coulomb equation $s = c + n \tan \phi$. The value of the normal pressure n in this case is given by equation (6), and is equal to $2cK\sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}}$. Therefore, the horizontal unit reaction on the rectangular face abcd, equivalent to the shearing resistance of the paving mixture along the two vertical sides, abhg and dcij, of the rectangular block under the loaded area, is given by

$$\frac{2\left(c + 2cK\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}\tan\phi\right)lt}{wt}$$

$$= 2\left(c + 2cK\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}\tan\phi\right)\frac{l}{w} \qquad (22)$$

The horizontal unit reaction on the rectangular face abcd equivalent to the maximum developed reaction of the wedge of pavement abcdef, immediately in front of the loaded area, is given by equation (8),

$$L = 2cJ\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}} \qquad (8)$$

The fourth source of pavement reaction to the braking stress listed was the pavement's tensile strength The Mohr diagram indicates that this tensile strength should be $2c\sqrt{\frac{1 - \sin\phi}{1 + \sin\phi}}$. Because of the possible presence of fine hair cracks, etc., which might often prevent the development of the full tensile strength of the pavement, it seems desirable to neglect the tensile strength as a source of pavement reaction to braking stresses

Therefore, the total horizontal unit reaction on the rectangular face abcd (Fig. 29) equivalent to the four sources of pavement reaction to braking stresses listed, is given by the summation of the quantities on the right hand sides of equations (21), (22), and (8), or

$$\frac{Q(c + V\tan\phi)l}{t} + \frac{2\left(c + 2cK\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}\tan\phi\right)l}{w} + 2cJ\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}} + \text{tensile strength} \qquad (23)$$

To prevent pavement failure under the combination of vertical tire pressure V and braking stress fV, tending to shove the pavement ahead of the loaded area (to the left in Fig. 29), the sum of the applied stresses must not exceed the sum of the reactions that can be developed by the pavement. This requirement for pavement stability is complied with when the sum of the effective applied stresses given by equation (19) is equated to the sum of the effective reactions developed by the pavement as indicated by equation (23). That is,

$$\frac{P(c + V\tan\phi)l}{t} + V\frac{1 - \sin\phi}{1 + \sin\phi} - 2c\sqrt{\frac{1 - \sin\phi}{1 + \sin\phi}}$$

$$= \frac{Q(c + V\tan\phi)l}{t} + \frac{2\left(c + 2cK\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}\tan\phi\right)l}{w} + 2cJ\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}$$

$$+ \text{tensile strength} \qquad (24)$$

Upon simplification, and neglecting the tensile strength, equation (24) becomes

$$V = c\left[\frac{2\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}\left(2K\frac{l}{w}\tan\phi + J + \frac{1 - \sin\phi}{1 + \sin\phi}\right) + 2\frac{l}{w} - \frac{l}{t}(P - Q)}{\frac{l}{t}(P - Q)\tan\phi + \frac{1 - \sin\phi}{1 + \sin\phi}}\right] \qquad (25)$$

Equation (25) is a stability equation for the design of bituminous pavements subjected to braking or acceleration stresses. While at first glance it may appear to be rather formidable, it is made up entirely of quantities that are either provided by the conditions of design assumed for any given project, or that can be determined experimentally in the laboratory.

The length l of the loaded area, and its width w, are established as soon as the wheel load and tire pressure are specified. The thickness of pavement t is usually designated more or less arbitrarily as 2 in., 3 in., etc. In equation (25), however, it should be observed that the quantities l, w and t occur only as the ratios l/w and l/t, which makes them easier to handle. An average value for l/w for airplane and many truck tires is 1.5, with a maximum range of 1.0 to 2.0. The value of l/t on the other hand might vary more widely depending upon the thickness of pavement and length of contact area. For pavements from 2 to 3 in. thick, the value of l/t might vary from 3 to 6 for truck tires, and might be as high as 10 or more for the largest airplane tires. Values of c and ϕ for any proposed bituminous mixture are measured by the triaxial test. P and Q are factors that are related to the maximum frictional resistance that can be developed between pavement and tire and between pavement and base, respectively. P and Q can be evaluated by field and laboratory tests and by utilizing equations (9) and (10), respectively. K and J are factors by which the unconfined compressive strength of the pavement must be multiplied to determine the lateral support L provided by the pavement adjacent to the loaded area in transverse and longitudinal directions, respectively. Representative values for K and J could be determined in the laboratory. In most cases, it would probably be quite conservative to assume that both K and J are equal to unity.

Figure 30 illustrates the application of equation (25) to the design of bituminous paving mixtures that are to be subjected to braking stresses. Values of $K = 1$, $J = 1$, $l/w = 1.5$, and $l/t = 4$ have been assumed, and the stability curves, therefore, reflect the influence of different values of $P - Q$ on the design of bituminous mixtures that must support a vertical load V of 100 psi., in addition to being subject to braking stresses. It will be recalled from equations (9) and (10) that neither P nor Q can have a value greater than unity. Consequently, a value of $P - Q = 1$ means that the value of Q must be zero, and indicates that there is no frictional resistance between pavement and base. This is obviously the most critical condition of design for a pavement subject to braking stresses, and is verified by the position of the curve labelled $P - Q = 1$ in Figure 30, which clearly requires bituminous paving mixtures with higher corresponding values of c and ϕ for a vertical load $V = 100$ psi., than any other stability curve on the chart. The stability curves in Figure 30 for successively smaller values of $P - Q$ indicate that bituminous mixtures with correspondingly smaller values of c and ϕ would be stable under braking stresses and a vertical load V of 100 psi. On the basis of equation (25), the stability curve for $P - Q = 0$ represents the condition of design where the frictional resistance between pavement and base is equal to the braking stress between pavement and tire. The stability curve for $P - Q = -1$, is the opposite extreme of the case where $P - Q = 1$, and represents the highly undesirable design condition of no frictional resistance between pavement and tire, although there is full development of the frictional resistance between pavement and base.

In connection with the development of equation (25) it was pointed out that there are two forces which tend to cause the pavement to shove ahead of the tire. One of these forces is the braking stress of the tire on the pavement, and the other is the tendency of the vertical load V to squeeze the bituminous mixture out from under the loaded area. The resistances that are mobilized to oppose one of these forces do not necessarily oppose the other. The resistances opposing the braking stress are the four previously listed.

1. Resistance of the wedge abcdef (Fig. 29) to forward displacement.

2. Frictional resistance between pavement and base, on plane adjg (Fig. 29).

3. Shearing resistance of bituminous pavement along the two vertical faces, abhg and dcij (Fig. 29).

4. Tensile strength of the pavement acting on the face ghij (Fig. 29).

However, under certain conditions of braking stress, only the first of these, that is, the resistance of the wedge abcdef (Fig. 29) to forward displacement can be mobilized to resist the tendency of the vertical load V to squeeze the pavement out from under the loaded area toward the front of the tire. The maximum horizontal resistance to forward displacement that can be mobilized by the

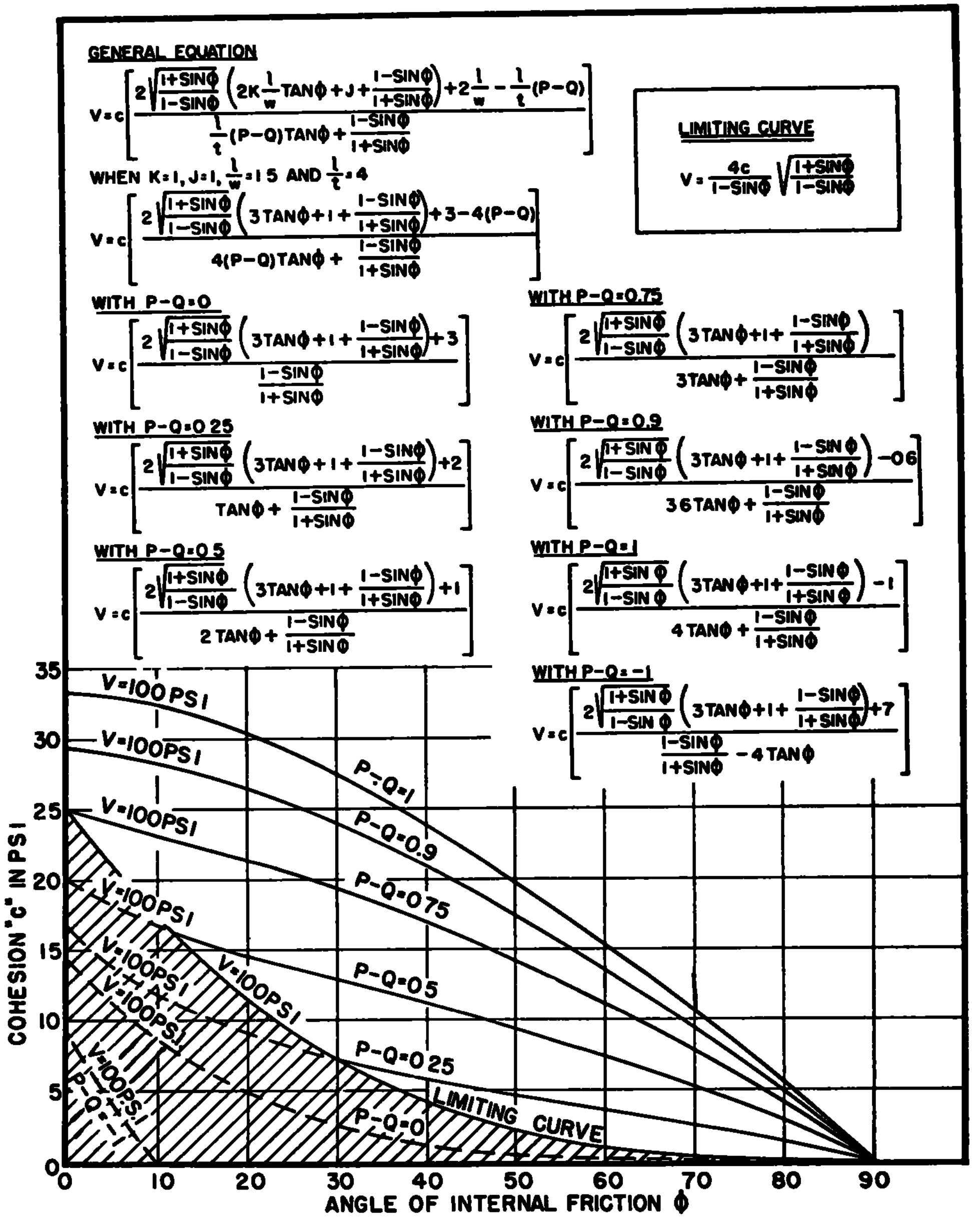

Figure 30. Illustrating Stability Curves for a Given Vertical Load for Bituminous Paving Mixtures subject to Braking Stresses

wedge of pavement abcdef is given by equation (8)

$$L = 2cJ\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}} \tag{8}$$

and the maximum vertical load V that can be applied to a bituminous pavement without causing it to be squeezed out in front of or behind the tire is obtained by substituting equation (8) in equation (3), which after simplification gives,

$$V = 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}} \times \left(\frac{J(1+\sin\phi)+(1-\sin\phi)}{1-\sin\phi}\right) \quad (26)$$

When $J = 1$, equation (26) reduces to equation (5)

$$V = \frac{4c}{1-\sin\phi}\sqrt{\frac{1+\sin\phi}{1-\sin\phi}} \quad (5)$$

When applying equation (25) to the design of bituminous mixtures that will be exposed to braking stresses, therefore, it must be kept clearly in mind that the pavement must at the same time always have sufficient stability to resist being squeezed out either ahead of or behind the tire, (or transversely) solely because of the vertical pressure V exerted by the tire on the loaded area. Consequently, there is a limit beyond which equation (25) cannot be employed for the design of bituminous pavements subject to braking stresses, because beyond this limit, equation (26) has become the more critical criterion of design. This is illustrated in Figure 30. All the $P - Q$ stability curves shown in this figure result from the use of equation (25). The stability curve labelled "limiting curve", on the other hand, results from the application of equation (26). For purposes of illustration in Figure 30, a value of $J = 1$ was assumed, which is probably quite conservative. For this value of J, equation (26) reduces to equation (5), which is the equation for the "limiting curve" shown in Figure 30.

Therefore, all bituminous mixtures having combinations of c and ϕ represented by points within the cross-hatched area of Figure 30, would be stable under the maximum braking stress for the value of the $P - Q$ curve through each particular point, but would tend to be squeezed out ahead or behind the tire by the applied vertical load $V = 100$ psi. Consequently, for the values of J, K, l/w and l/t specified for Figure 30, only those bituminous mixtures with corresponding values of c and ϕ represented by points either on or to the right of the curve labelled "limiting curve" would have the stability needed to avoid being squeezed out to the front or rear of the tire by the vertical load V, and to resist the tendency to displacement caused by the maximum braking stress applied. To the right of this limiting curve the corresponding values of c and ϕ required would depend upon the particular $P - Q$ curve indicated as the criterion for design, and only bituminous mixtures whose c and ϕ coordinates were either on or to the right of this particular $P - Q$ curve would be stable under both the maximum braking stress and the vertical load V.

It should be clear that stability diagrams similar to Figure 30, based on equation (25) for other combinations of values for J, K, l/w and l/t, could be easily prepared. It should also be noted, that for the particular values for J, K, l/w and l/t employed for Figure 30, corresponding stability curves for any other value of vertical load V can be obtained on the basis of simple proportionality. Suppose, for example, that the corresponding curve was desired for $P - Q = 1$, when $V = 50$ psi. For all values of ϕ along the abscissa, points marking off one-half the ordinate between the abscissa and the curve labelled $P - Q = 1$ for $V = 100$ psi. would be shown. The curve through these points would be the required curve for $P - Q = 1$, when $V = 50$ psi. Similarly, if for all values of ϕ along the abscissa, points marking off twice the value of the ordinate between the abscissa and the curve labelled $P - Q = 1$ for $V = 100$ psi. are drawn, the curve through these points will represent the curve for $P - Q = 1$ when $V = 200$ psi. This procedure of simple proportionality for obtaining stability curves for other values of V than those for $V = 100$ psi, shown in Figure 30, is possible because it will be observed from equation (25) that cohesion c is a multiplier for the balance of the quantity on the right hand side of the equation. Consequently, if every factor on the right hand side is constant except c, the value of V depends directly on the value of c, and vice versa.

A similar procedure can be employed to obtain additional stability curves for nearly all of the other stability diagrams contained in this paper (e.g. Figs. 14, 17 and 27). It will be observed that like equation (25), the stability equations on which each of these stability diagrams are based, have cohesion c as a multiplier for the remainder of the equation on the right hand side.

CRITICAL CONDITIONS FOR DESIGN

In the introduction it was pointed out that there are three principal conditions of pavement stability to be considered when designing bituminous paving mixtures.

1. Stability under stationary loads
2. Stability under loads moving at a relatively high but uniform rate of speed.
3. Stability under braking and accelerating stresses of traffic.

Practical field observation has indicated that pavement instability is usually of three different types.

1. Squeezing out of the pavement from under the wheel in a transverse direction for both stationary and moving loads.
2. Shoving of the pavement at bus stops, traffic lights, etc., probably caused by braking accelerating stresses, and the development of of washboard occasionally in unstable pavements carrying moving traffic.
3. Actual tearing of the pavement under moving traffic, generally when a relatively thin layer of pavement has been placed on a base to which it is either bonded poorly or not at all

The stability conditions for stationary wheel loads are indicated in Figure 27 for a vertical load V of 100 psi. The minimum values of c and ϕ required for stability are seen to depend on the magnitude of $P + Q$. The values of P and Q in turn depend upon the amount of frictional resistance developed between pavement and tire and between pavement and base, respectively. Therefore, Figure 27 emphasizes the importance of specifying a construction procedure that will ensure the maximum bond between the pavement and base, giving a high value for Q Figure 27 also stresses the value of designing bituminous mixtures capable of developing a high frictional resistance between pavement and tire, that is a high value of P Consequently, Figure 27 demonstrates that if the values of both P and Q can be kept high, bituminous mixtures with much lower values of c and ϕ than would otherwise be necessary, will provide adequate stability to carry the applied load without failure.

With regard to the design of pavements subject to braking stresses Figure 30 demonstrates the advantages of keeping the value of $P - Q$ as low as possible. For safety reasons it is desirable to design and construct bituminous pavements having a very high coefficient of friction between pavement and tire.

This in turn tends to provide a high value for P Since the value of P should be high for good design, the desired low value for $P - Q$ can only be obtained if the value of Q is also high. That is, construction procedure should be specified that will ensure a strong bond between pavement and base. It is quite apparent from Figure 30 that if the frictional resistance between pavement and tire is high, and if a strong bond is obtained between pavement and base, resulting in a low value for $P - Q$, bituminous mixtures with much lower values of c and ϕ than would otherwise be necessary will have the stability required to avoid failure under any given applied load.

Equation (25) indicates the importance of both pavement thickness and adequate bond between pavement and base when designing for pavement stability to resist braking stresses. When resurfacing over an old pavement, or when building a new pavement over a smooth base, the bond between old and new pavement, or between pavement and base, may be rather weak unless adequate care is taken. If the old pavement or the base is uneven, the new pavement may be relatively thin in some areas. In equation (25) it will be noted that both the terms l/t and $P - Q$ subtract from the value of the numerator and also appear in the denominator. Therefore, for a bituminous pavement with given values of c and ϕ, any factors that increase the values of either l/t or $P - Q$, or both, other variables being constant, will decrease the stability of the pavement under braking or accleration stresses. The length of contact area l, is constant for any given wheel load and tire pressure. Therefore, l/t will increase as the thickness t of the pavement is decreased. The value of $P - Q$ tends to increase as the value of Q is decreased, that is, as the bond between pavement and base becomes weaker. Consequently, equation (25) demonstrates that the resistance of a bituminous pavement to braking and accleration stresses is improved by adequate thickness and by taking proper measures to obtain a strong bond between pavement and base.

The position of the limiting curve in Figure 30 shows that if the bond between pavement and base is very strong, and approaches the shearing resistance of the bituminous mixture itself, (low value for $P - Q$), the tendency of the tire load V to squeeze the pavement out from under the wheel may be a more critical condition of design than the action of the braking stress. That is, Figure 30 indicates that under these particular conditions, the stresses imposed by given vehicles moving at a uniform rate of speed may represent a more severe stability requirement for bituminous mixtures than the maximum braking stresses applied by the same vehicles

It is a very difficult matter to determine whether a more conservative design results when based upon moving rather than upon stationary loads or vice versa. Any such comparison is complicated by the fact that the value of cohesion c developed by any given pavement to resist the stresses of moving loads is very much greater than the value of cohesion c developed by the same pavement under a static load. This was demonstrated by the data of Figures 18(b) and 19. It is also possible that the value of the angle of internal friction ϕ is somewhat different under moving than under static loads. Therefore, until more quantitative data becomes available concerning the influence of rate of loading on the values of cohesion c, and angle of internal friction ϕ, and this information has been correlated with field performance, the stability requirements for moving versus static loads cannot be accurately compared. Experienced bituminous engineers have observed pavements that are stable under moving vehicles, but into which the wheels of the same vehicles would rapidly settle if they stopped. This is sometimes observed on freshly constructed bituminous pavements. On the other hand, a thin bituminous pavement that is poorly bonded to the base or layer of pavement below, may tear under moving loads within a short time, although apparently stable under static loads. Consequently, it seems quite doubtful that any general statement to the effect that stationary loads always represent a more critical condition of design for bituminous pavements than moving loads could be justified. It appears on the other hand that there are conditions for which moving loads represent a more serious criterion of design than stationary loads, and other conditions for which stationary loads represent the more severe design requirement

For the case of pavement stability at bus stops, traffic lights, and other areas where vehicles may come to a full stop, the question arises as to whether the pavement for these sections should be designed for moving or for stationary loads. During most of the period when the brakes are bringing the vehicle to a full stop, the value of cohesion c developed in the pavement may be many times greater than its magnitude under a stationary load. On the other hand, for a very brief interval just before the vehicle stops, the rate of movement becomes so low that the value of cohesion c developed may not be much larger than for a stationary load. Consequently, if there is a rather weak bond between pavement and base at an area subject to much stopping and starting of traffic, the design curve indicated by one of the appropriate larger values of $P - Q$ in Figure 30 should be selected as the basis for design, using a low rate of strain when testing the bituminous mixture in the triaxial test However, if an excellent bond between pavement and base is assured, pavement design for bus stops, traffic lights, and other areas where there is much starting and stopping of traffic, should probably be based upon the condition of stationary loads

For pavements subject to traffic moving at relatively high rates of speed, and where, although braking stresses may be applied, they are used only to obtain temporary deceleration and do not bring vehicles to a full stop (e g normal highway traffic in rural areas and the runways at airports) it would seem reasonable to base bituminous mixture design on moving loads That is, a relatively high rate of strain would be used when testing the paving mixture in triaxial equipment, in order that a value of cohesion c for the mixture might be obtained in the laboratory for design purposes, that would approximate the value of cohesion c developed in the resulting pavement under moving traffic. Further experimental work is required to determine what the permissible rate of strain should be for the triaxial testing of bituminous mixtures that are to carry moving wheel loads

For pavements subject to stationary loads, or to very slowly moving traffic, bituminous mixture design should probably be based upon static loads. The pavements for most city streets, all parking areas, and the aprons, taxiways, and turnaround areas at the ends of runways at airports, are representative examples. A very low rate of strain should be used for triaxial tests on bituminous mixtures that are proposed for pavements in these areas

Finally, until there has been an opportunity to build up information on the field performance of bituminous mixtures designed by the rational method and employing the triaxial test, it would seem prudent to base the the design of bituminous mixtures on the stationary load condition, for the maximum pressure applied to the contact area (*14*, *15*), and possibly including an impact factor (*23*). *If particular care is taken to obtain a strong bond between pavement and base*, it is believed that the curves of Figure 14, for which $K = 1$, would provide a conservative basis for design. Later, as confidence in this method of design becomes established, and as more experimental data become available, the refinements indicated in the paper, particularly with regard to Figures 17, 27 and 30, which might lead to a less conservative design for both static and moving loads, could be gradually adopted.

RELATIONSHIPS BETWEEN UNCONFINED COMPRESSION AND TRIAXIAL TESTS

Several different forms of the unconfined compression test have been proposed from time to time as methods for measuring the stability of bituminous mixtures. The most recent development of this test to obtain some prominence is the Marshall stability test.

It has been the principal purpose of this paper to indicate how the triaxial test can be employed to measure the strengths of bituminous paving mixtures under conditions of stress similar to those to which they are subjected by traffic in the field. However, this test is rather time consuming, and it would be highly advantageous if some simpler and more rapidly performed test could be correlated with the triaxial test and used for the checking and control of the stability of bituminous mixtures in the field Since the unconfined compression test appears to be of the same general type, it is worth while to determine whether there is any useful relationship between this test and the triaxial test that could be employed for this purpose Figure 31 illustrates the discussion of this point, which demonstrates that the unconfined compression test can be quite misleading, insofar as indicating the stability that bituminous paving mixtures can develop in the field is concerned.

In Figure 31(a) the Mohr envelopes xw, yv and zu represent three different bituminous mixtures with corresponding values for cohesion c and angle of internal friction ϕ, $c_1\phi_1$, $c_2\phi_2$, and $c_3\phi_3$, respectively. Similarly, in Figure 31(b) the Mohr envelopes lr, mq and np represent bituminous mixtures with corresponding values for cohesion c and angle of internal friction ϕ, $c_4\phi_4$, $c_5\phi_5$, and $c_6\phi_6$, respectively.

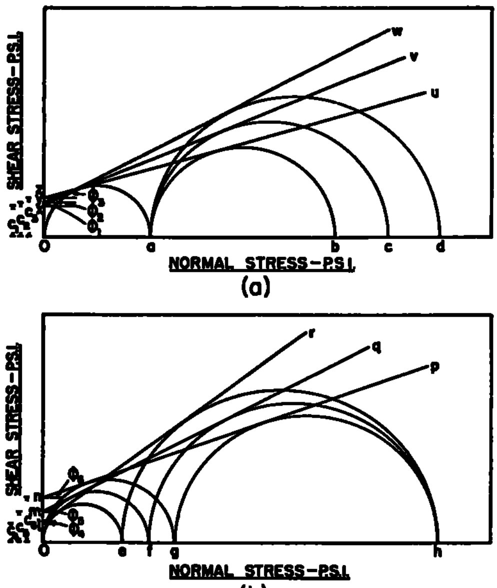

Figure 31. Demonstrating the Inadequacy of an Unconfined Compression Test for measuring the Stability of Bituminous Paving Mixtures

Figure 31(a) indicates that the unconfined compressive strength Oa is exactly the same for each of the quite different bituminous mixtures represented by Mohr envelopes xw, yv and zu. For an unconfined compressive strength test, the amount of lateral support provided is zero. On the other hand, bituminous pavements in place in the field are able to mobilize lateral support from the pavement adjacent to the loaded area. In an earlier part of this paper, it was indicated that the unconfined compressive strength was a measure of this lateral support. Consequently, the stability under service conditions of the three bituminous mixtures represented by Mohr envelopes xw, yv and zu is indicated by Mohr circles ad, ac, and ab, respectively. If traffic loads exert a vertical pressure V equal to point d in Figure 31(a), then only the bituminous mixture represented by the Mohr envelope xw would be stable in the field, and the other two represented by Mohr envelopes yv and zu would be unstable It should be noted again that the unconfined compression test would give all three of these bituminous mixtures exactly the same stability rating, Oa. Consequently, Figure 31(a) demonstrates that the unconfined compression test may indicate exactly the same stability for bituminous mixtures that would have widely different stabilities under field conditions.

Figure 31(b) represents the reverse of the situation illustrated in Figure 31(a). The unconfined compressive strength of the three bituminous mixtures lr, mq and np are indicated by Oe, Of and Og, respectively. Since the unconfined compressive strength is a measure of the lateral support provided by the pavement adjacent to the loaded area, Figure 31(b) demonstrates that these three bituminous mixtures are all capable of developing exactly the same resistance Oh to the applied vertical load, under the conditions that exist in the field. Nevertheless, if the stability of these three mixtures were evaluated by the unconfined compression test, three widely different stability ratings Oe, Of and Og would be indicated. If the vertical load V to be carried is equal to Oh, the triaxial test would indicate that the three bituminous mixtures had the same stability, and that all three would be satisfactory. If the minimum stability requirement according to the unconfined compression test were given by Og, then this test would indicate that only the bituminous mixture represented by Mohr envelop np would have the necessary stability under field conditions and that the mixtures represented by Mohr envelopes lr and mq would be unstable. Therefore, Figure 31(b) shows that the unconfined compression test could reject as being unstable bituminous paving mixtures that would develop quite adequate stability under field conditions.

Figure 31 clearly demonstrates, therefore, that the unconfined compression test, and all similar tests, are fundamentally incapable of providing trustworthy measurements of the stability that bituminous paving mixtures can develop under field conditions. Figure 31(a) shows that the unconfined compression test is capable of wrongly indicating adequate stabilities for bituminous mixtures that would be unstable in the field Figure 31(b) demonstrates that the unconfined compression test is also capable of rejecting bituminous mixtures that would have adequate stability under field conditions.

It is sometimes stated in defense of unconfined compression and other similar empirical stability tests that they are capable of reflecting changes in aggregate gradation, filler content, bitumen content, etc. Figure 31(a) demonstrates that such changes could be made in the composition of bituminous mixtures which would have little or no effect on their unconfined compressive strength, but that would at the same time seriously influence the stability of the pavement under the conditions of stress to which it is exposed in the field. On the other hand, Figure 31(b) indicates that wide changes can be made in the composition of bituminous mixtures, which would cause large variations in their unconfined compressive strength values, but that would have little influence on pavement stability under field conditions. Consequently, it is apparent from Figure 31 that the unconfined compression test either may or may not be influenced by wide changes in the composition of bituminous mixtures, which are reflected by large variations in their c and ϕ values. Furthermore, the influence on the unconfined compression test values caused by these changes in composition, may lead to quite erroneous conclusions concerning the influence of these changes on pavement stability under field conditions.

There seems to be little doubt that engineers can be led seriously astray when endeavouring to measure the stability of bituminous mixtures by the unconfined compression test, by any of the variations of this test, or by any stability test that can be correlated with the unconfined compression test. Figure 31 indicates that these tests may actually be so misleading that it may be questioned whether they serve any useful purpose as stability tests. The other empirical stability tests in common use at this time should also be suspect, unless it can be shown that they meet the fundamental requirements that any satisfactory stability test for bituminous paving mixtures must possess

At the present time, we know of no test that can provide a satisfactory measure of the stability of bituminous pavements, except the triaxial test. While it is rather complex and time consuming as currently performed, it seems to be capable of furnishing data from which the actual stability of bituminous mixtures under field conditions can be calculated, and on which a rational method of design for the strength of bituminous pavements can be based. Future research may develop a simple rapidly performed test, useful for field control, that can be closely correlated with the triaxial test.

GENERAL

In this section a few general comments will be made that could not be included elsewhere in this paper

1. Some engineers may feel that the mathematical development required for the rational method of design based upon the triaxial test, outlined in this paper, is a serious barrier to the practical routine use of this method. Such, however, is not the case. When using a camera, for example, even a movie camera, we seldom concern ourselves about the mathematics through which someone had to work in order to design the camera. We merely make the proper adjustments for distance, lens opening and shutter speed, and proceed to take the picture.

In the case of the rational method of design for bituminous mixtures outlined here, the finished design charts correspond to the camera. If agreement can be reached concerning the mathematical basis of these charts, and pertaining to the rate of strain at which the triaxial test is to be run to provide the appropriate values for c and ϕ for the bituminous mixtures proposed for any given project, we can disregard the mathematical development itself when using the charts. The values of c and ϕ obtained from the triaxial test, provide the coordinates of a point on the appropriate chart. The position of this point relative to the particular stability curve specified by the design requirements for the project, indicates whether the bituminous mixture tested has the required stability or not.

2. Triaxial tests must be made on briquettes of bituminous mixtures formed in the laboratory. It is highly important that the bituminous mixture in the compacted briquette for the triaxial test should duplicate as nearly as possible the structure that the same bituminous mixture will attain in the road surface. Endersby (*1*) and Hveem (*3*) have provided a very fine account of the work that has been done, and that is still going on, to obtain a laboratory compaction device that will provide samples for the triaxial test having a structure identical with that developed under rolling and traffic in the field. The development of such a compaction test is of the greatest importance since stability measurements on a laboratory sample are of questionable value unless the laboratory sample duplicates the characteristics and structure that would be developed in the field by the bituminous mixture being tested

The comprehensive investigation of the design of bituminous mixtures conducted by the U. S. Corps of Engineers (*18*) has provided some excellent quantitative data on the increase in the density of bituminous pavements that occurs under traffic. Their studies have shown that the air voids content of what might ordinarily be considered to be well designed bituminous mixtures may approach zero with the increased density resulting from traffic, and serious loss of stability may occur.

In the triaxial testing of a bituminous mixture, therefore, it would seem advisable to determine its stability at the density the pavement will have when rolling is complete, and also at the ultimate density it may be expected eventually to obtain under traffic, since both conditions may be critical in the life of the pavement.

3 The dimensions of the specimen tested in triaxial compression must receive careful consideration, if representative values for cohesion c, and angle of internal friction ϕ, are to be obtained.

To avoid the effects of friction between the end plates and the test briquette during loading, and the direct transfer of load between the two plates, the height of the specimen should be at least twice its diameter. If the angle of internal friction ϕ is likely to approach 40 to 45 deg , the height should be at least two and one-half times the diameter. Nijboer (*10*) recommends a height to diameter ratio of three for asphaltic concrete mixtures, and a ratio of two and one-half for sand asphalt, including sheet asphalt mixtures

The diameter of the test briquette should be not less than four times the diameter of the largest particles in the bituminous mixture. Smith (*4*) reports excellent reproducibility of test results for this ratio. Nijboer (*10*), on the other hand, prefers that the diameter of the test specimen should be at least six times the diameter of the largest particles. He points out that in this case the cross-sectional area of a single large particle is only about 3 percent of the cross-sectional area of the test briquette.

4. In the development of the rational method of design for bituminous paving mixtures outlined in this paper, no mention has been made of the influence of the repeated loadings of highway or airport traffic on design.

This is a matter that may have to be answered by practical observations of the field performance of bituminous mixtures designed by this method.

On the other hand, the Corps of Engineers' investigation (*18*) indicated that one of the effects of continued traffic on bituminous pavements was a very marked increase in density As long as this density increase does not eventually reduce the air voids below the critical value (normally about 2 to 3 percent by volume) at which stability usually begins to decrease, the influence of repeated traffic loads should ordinarily be beneficial, and should provide an increase in stability rather than otherwise. Consequently, it may be that as long as continued traffic does not increase the pavement density to the point where the volume of air voids becomes critically low, no provision in design for the repeated loads of traffic may be necessary. In addition, it is well known that bituminous binders tend to harden with time, and to develop an internal structure of their own. This gradual hardening of the binder should increase the stability of bituminous pavements with time, and may be adequate in itself to compensate for the effect of repeated traffic loads on pavement stability, if this is an item that must be taken into account.

In view of these considerations, it may not be unreasonable to ask, whether a bituminous pavement that eventually shows evidence of distress under the repeated traffic of vehicles with some maximum wheel load and tire pressure, was actually designed for that magnitude of load in the first place? It may have been underdesigned from the beginning

Nevertheless, if in spite of these observations, field experience should indicate that repetitions of traffic loads must be considered in connection with the stability of bituminous pavements, probably they could satisfactorily be taken into account by multiplying the design load by a traffic factor. This would be equivalent to employing a safety factor.

5. Figures 27 and 30 illustrate the very important fact that in addition to the inherent stability of the bituminous mixture itself, the stablity of a bituminous pavement in service depends very substantially on the frictional resistance between pavement and tire and between pavement and base. Therefore, a bituminous mixture indicated to be highly stable according to the present commonly used empirical stability tests, such as Hubbard-Field, Marshall, Hveem, etc. may actually show unsatisfactory stability in the field because of low frictional resistance between pavement and tire and between pavement and base. Another bituminous mixture, rated to have questionable stability by these empirical tests, may develop high stability in the field because of high frictional resistance between pavement and tire and between pavement and base. In addition, it is entirely beyond the capacity of the present empirical tests to indicate the influence of pavement thickness on the stability of a bituminous pavement in the field. Therefore, in addition to their other serious deficiencies, the present commonly used empirical tests are quite unsatisfactory for measuring the stability of bituminous pavements in service, because they are fundamentally incapable of taking into account the important contributions to pavement stability of the frictional resistance between pavement and tire and between pavement and base, and of pavement thickness. This fact alone emphasizes the need for the development and general adoption of a rational method of design for bituminous pavements.

CONCLUSION

Because of the fundamental inadequacy of the present commonly used empirical methods for measuring the stability of bituminous pavements, and for expressing their strengths on a pounds per square inch basis, an attempt has been made in this paper to outline a rational approach to this problem. It is realized that this presentation may have over-simplified the solution at some points, and that further refinements may be desirable. It may have disregarded certain factors that should be included, and may have barely mentioned others that should receive more emphasis. On the other hand, space limitations prevent reference to every detail, and the paper has been confined to a discussion of what appear to be the more important principles requiring consideration. It is appreciated that there may be quite other approaches that could provide either equally or more accurate solutions, and it is hoped that the search for these will be stimulated.

Nevertheless, the present paper has outlined a rational method for designing the strength of bituminous pavements on a unit strength basis. Among other items, it has indicated how the amount of lateral support L provided by the pavement adjacent to the loaded area may be evaluated. It has reviewed the influence that the rate of strain selected for a triaxial test may have on the value of cohesion c, and possibly on the value of the angle of internal friction ϕ, determined for a bituminous mixture. It has indicated in a quantitative manner, the importance of the frictional resistance between pavement and tire and between pavement and base, and of pavement thickness on the stability of bituminous mixtures in service.

It is one of the most serious criticisms of the present commonly used empirical tests, that they are entirely incapable of taking these very important factors into account. It has pointed out that different designs may be justified for pavements for moving than for stationary traffic, and the basis on which these differences in design may be specified. The stability equations and stability diagrams included, indicate quantitatively the direction in which the cohesion c and angle of internal friction ϕ for bituminous mixtures must be varied to obtain either greater or less stability. Finally, the stability equations and diagrams show that it should be quite possible to design bituminous pavements for either airports or highways, that would be quite stable under the heaviest wheel loads, and with tire pressures up to 300 or 400 psi. or more. However, they also indicate that the design of bituminous mixtures for these high tire pressures, can be expected to require considerably more care than has been given to this matter for the much lower tire pressures in use to-day

The current commonly used empirical tests are unable to provide the fundamental information required for the quantitative evaluation of any one of the items included in the previous paragraph. Nevertheless, in spite of their fundamental inadequacies, these empirical tests are likely to be favored by their advocates for a long time to come, chiefly because of their simplicity, and regardless of the fact that simplicity of test procedure is a doubtful virtue, if the test results obtained are of questionable value for the purpose intended. Consequently, the development and general introduction of a rational method of design for bituminous paving mixtures, will probably be just as controversial a topic during the next few years, as the problem of the overall thickness of flexible pavements has proven to be.

SUMMARY

1. The fundamental inadequacies of the empirical tests commonly used for the design of bituminous mixtures at the present time are mentioned, and the necessity for the development of a rational method of design, which would evaluate the strengths of bituminous mixtures on a pounds per square inch basis, is pointed out.

2. The open triaxial test, which is widely used on this continent, is described, together with the method of plotting the triaxial data in the form of a Mohr diagram.

3. From the geometrical and trigonometrical relationships of the Mohr diagram, a general equation for the stability of bituminous mixtures is developed.

4. A method for evaluating the lateral support L provided by the pavement adjacent to the loaded area is outlined.

5. A description is given of the use of the cell triaxial test for determining the influence of the viscous resistance of a bituminous mixture on its stability, and for measuring the effect of the rate of strain on the value of the cohesion c, and possibly of the angle of internal friction ϕ, obtained for a bituminous mixture.

6. The increase in the stability of a bituminous pavement due to frictional resistance between pavement and tire and between pavement and base, is indicated.

7. The influence of braking and acceleration stresses on the design of bituminous pavements has been considered.

8 It is pointed out, that in addition to their other serious deficiencies, the present commonly used empirical tests, such as Hubbard-Field, Marshall, Hveem, etc., are unable to take into account the important contribution to pavement stability made by the frictional resistance between pavement and tire and between pavement and base, and the influence of pavement thickness on pavement stability.

9. It is demonstrated that the unconfined compression test, and all similar tests, can be quite misleading, insofar as indicating the stability that bituminous mixtures can develop in the field is concerned

10 Stability equations and illustrative diagrams for different conditions of bituminous pavement design are included.

11. A brief discussion of the critical conditions of stability to be considered for the design of bituminous pavements for both moving and stationary loads, is included.

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DISCUSSION

WALTER C. RICKETTS, *Office of the Chief of Engineers*—Dr McLeod has made an excellent presentation of a theoretical method for the design for bituminous paving mixtures based on the triaxial test. The description of the triaxial apparatus and the references to investigators of the subject indicate that Dr. McLeod refers to the triaxial machine and method described in two papers[1, 2] by V. R. Smith, presented in 1949 before the Association of Asphalt Paving Technologists and American Society for Testing Materials or similar method based on the triaxial test.

It is considered that the paper merits discussion for the following reasons:

1. The possibility that the casual reader may infer that Dr. McLeod's rational method for design of bituminous pavements is now ready for immediate use by the design and construction engineer and the numerous charts and equations are technically correct, fully supported by engineering data, and adequately correlated with pavement performance in the field.

2. The criticism that all existing testing machines and methods for designing bituminous pavements, particularly Hubbard-Field, Marshall, and Hveem machines and design methods are inadequate, misleading, and confusing even to experienced design engineers.

The purpose of this discussion is to show that:

1. Considerable laboratory work and correlation with pavement performance is required before a method of design for bituminous pavements based on the triaxial test could be used with confidence by the design and construction engineer.

2. Dr. McLeod's criticism regarding the inadequacy of existing machines and methods, particularly Hubbard-Field, Marshall, and Hveem, to properly design bituminous pavements is unwarranted and is not substantiated by experience.

3 Information for design of bituminous pavements taken from Dr McLeod's figures and equations may be misleading and technically incorrect due to the many assumptions unsupported by engineering data used in their formulation.

PRESENT STATUS OF TRIAXIAL TESTING OF BITUMINOUS PAVING MIXTURES

A committee known as the "Triaxial Institute" composed of representatives of the State Highway Departments of Washington, Oregon, and California, Universities of Washington and California, California Research Corporation, Shell Development Company, headed by Mr. V. A. Endersby, Shell Development Company, Emeryville, California, was formed to conduct research work and develop a suitable method for designing bituminous pavement based on the triaxial test.

There are listed below quotations from preprints of two papers by Messrs. Endersby and Smith, both members of the Triaxial Institute, presented at recent A S T.M. meeting in San Francisco. California.

[1] V R Smith, "Triaxial Stability Method for Flexible Pavement Design," *Proceedings* AAPT, Vol. 18 (1949).

[2] Preprint, V. R. Smith, "Application of the Triaxial Test to Bituminous Mixtures," ASTM, San Francisco, Calif, October (1949).

Paper[3] *by V. A. Endersby*

1. "The reasons for the formation of the Institute are discussed and research of the fundamental problem of forming test specimens to properly represent materials laid in pavements is described."

2. "*Outstanding Questions.* There are various outstanding questions regarding the triaxial test that need further investigation:

a. What is the most acceptable interpretation for use by the routine testing engineer?

b. What is the effect of low height/diameter ratio prevalent in the field under road loading conditions?

c. What is the real meaning of the intercept which denotes static or initial resistance in the Mohr diagram? This is usually called "cohesion" but shows some properties not compatible with the definition, such as in some mixes decreases in its value with decreasing friction.

d What is the proper method for compacting samples? (This point is so important that we devote a special section to it)"

3. "Basically the Institute was formed with two objects: First, to determine to the satisfaction of a group of representative interests the fundamentals of a test which would be fully scientific in principle and whose results could be quickly interpreted in terms suitable for routine laboratory; second, to bring about as complete correlation as possible with other tests, it being well recognized that long established tests will not be readily abandoned even though scientifically superior ones can be established."

4. "*The Problem of Realistics Test Specimens* The first meeting brought out a definite consensus that a very primary problem in the general field has not been solved. That is, most current methods of fabricating test specimens did not reproduce the properties of field mixes."

5. "We believe that any method, however inexpensive, which produces a material for testing which is radically different from that actually placed in the field is unacceptable. The Committee departs from the widely held view that the testing method is the thing and that methods of fabrication are more or less incidental and unimportant While our co-operative physical research has not proceeded far as yet, we can show very striking evidence in results from the laboratories of some of the members of the Committee "

Paper[2] *by V. R. Smith*

1. "All stability tests now in use, including triaxial, will yield differences in measured stability values when different specimen compaction procedures are employed. This matter deserves maximum emphasis."

2. "Kneading type compaction is known to yield specimens approximating closely the particle orientation and stability properties obtained in actual field construction This matter certainly deserves further study and such studies now are being undertaken by various interested groups."

3. "Figure 4[4] shows the results of tests on hot plant mixes, road mixes (both machine mixed and blade mixed) and sheet asphalt mixes employed by various highway departments throughout the United States All of these mixes were made strictly according to the specifications and construction practices of these highway organizations and are considered satisfactory for the traffic conditions encountered The traffic conditions represented vary from moderate with the road mixes to heavy with the sheet asphalts and hot plant mixes. Except in a few instances first hand observation of the performance properties of these surfaces has not been possible."

4. "Experience with the evaluation charts presented in Figures 4 and 5 is limited. Possibly these charts will have to be revised after more extensive data on service behavior are obtained."

5. "*Note:* (Forming of test specimens having properties closely simulating those obtained in field construction is essential in procuring precise triaxial test results. At the present time several road material laboratories have under development improved briquet compaction procedures which promise to supersede current methods.)"

It is believed that the following comments

[3] Preprint, V A Endersby, "The History and Theory of Triaxial Testing and the Preparation of Realistic Test Specimens," a report of the Triaxial Institute ASTM, San Francisco, Calif , October (1949).

[4] Figures 4 and 5 indicate mixes considered to be satisfactory and unsatisfactory for heavy traffic and light traffic based on values of c, unit cohesion, and ϕ, angle of internal friction

are in accord with the quotations mentioned above:

1. The Triaxial Institute recognizes the need for research on a number of problems in connection with triaxial testing of bituminous mixtures and is planning the necessary experimental work to solve the problems.

2. The problem of forming test specimens to represent properly materials laid in pavements is a high priority item as most of the current methods do not produce test specimens comparable with field mixtures.

3. It will be necessary to formulate an interpretation of the results obtained by triaxial testing suitable for use by the routine testing engineer.

4. The work performed to date by Institute members indicates that triaxial test values on specimens prepared by different compactive methods are at variance.

5. The correlation of triaxial values for unsatisfactory and satisfactory bituminous mixes as recommended by Mr. Smith with service behavior of actual pavements is limited and his Figures 4 and 5 are subject to change when more extensive data on service behavior are obtained.

It is to be emphasized that this discussion is not intended to be critical of the Triaxial Institute or any experimental work on triaxial testing accomplished by its members or other investigators in connection with bituminous pavements. The objectives and the type of research planned by the Triaxial Institute as described in Mr. Endersby's paper are considered to be above criticism and technically sound for developing a design method for bituminous pavements based on the triaxial test.

ANALYSIS OF DR. McLEOD'S EQUATIONS AND FIGURES

A careful study of the numerous equations and figures indicates that many assumed values unsupported by engineering data have been used in the formulation of the equations and design charts. Some of the assumptions are of such a magnitude as to seriously affect the validity and possible usefulness of the equations for design purposes while other assumed values are numerically small and their effect is proportional to their size

In this discussion no attempt is made to follow completely all of these unsupported assumptions through all of the various equations and figures presented Only a relatively small number of unsupported assumptions which are considered to render the equations and charts inadequate at present time for use in designing bituminous pavements are discussed.

The rational method of design for bituminous paving mixtures proposed by Dr. McLeod is based upon triaxial testing which the Triaxial Institute recognizes as a method of test in need of considerable development and correlation with pavement performance before it could be of use to design engineers charged with the responsibility of providing adequate pavements.

One assumption used is the symbol L which represents the amount of lateral support in pounds per square inch developed by the portion of the pavement adjacent to the loaded area Later it is indicated that the value for L is equal or greater under some conditions to the unconfined compressive strength of the pavement. There are no experimental engineering data offered to support this claim. Further, no explanation is given as to what the value for L might be for various thicknesses of pavement or for various pavement temperatures

Figure 11, which is based on equation 3 with value of L of 30 psi., indicates that all paving mixtures to the right of curve labelled $V = 100$ psi., $L = 30$ psi, would have required stability and those to the left would be unsatisfactory. Here again is an assumption as no engineering data are offered to substantiate the fact that all mixtures considered satisfactory possess an L value of 30 psi. Further, no evidence of correlation with pavement performance is given which would indicate that all paving mixtures to the right of the curve are satisfactory for a V of 100 psi.

Because of certain factors not taken into consideration regarding the assumption that the lateral support L is equal to the unconfined compressive strength of the pavement, Figure 16 is offered to show the lateral support is somewhat greater than the unconfined compressive strength. Although Dr McLeod admits that in an actual pavement the wedge of the material under stress just outside the loaded areas does not necessarily conform to Figure 16a, shearing resistance values of the triangular end sections are assumed which may vary from 10 to 50 percent of the shearing resistance provided by diagonal plane ABCD depending on size of tire. Further, the additional lateral support thus provided may be 10 to 50 percent more than the unconfined compressive strength of the pavement. Again the values are assumed and not supported by engineering data

An additional source of lateral support L beyond the unconfined compressive strength of the pavement is introduced into equations 6 and 7 by multiplying the unconfined compressive strength by a factor K. Again no engineering data are given as to what the value of K might be. Figure 17 is based on assuming varying values of K ranging from 0 to 10

It is pointed out by Dr McLeod that values of cohesion c shown in Figure 17 may not be adequate in all cases and should be determined by field performance of bituminous mixtures with known values of c and ϕ

Among other factors represented by symbols which are used in the preparation of the various figures and formulas are f, coefficient of friction between pavement and tire, and g, coefficient of friction between pavement and base Again engineering data regarding values of f and g to be used in the proposed design have not been furnished.

No attempt has been made to list all of the equations and figures in which various symbols unsupported by engineering data have been introduced.

HISTORY OF HUBBARD-FIELD, MARSHALL, AND HVEEM TESTING MACHINES AND METHODS

Hubbard-Field Testing Machine—During 1926 and 1927, Messrs Hubbard and Field of the Asphalt Institute conducted extensive investigational work to develop a practical testing machine for use in designing sand type asphalt paving mixtures. Subsequent investigational work resulted in modifying the machine so it was adaptable for use in designing asphaltic concrete mixtures.

The application of the Hubbard-Field machine to design is contained in The Asphalt Institute publication, "Research Series No. 1, The Rational Design of Asphalt Paving Mixtures."

The Hubbard-Field machine and the principles of design as outlined by Mr. Hubbard have been used by design engineers for the past twenty years for designing paving mixtures used in constructing many miles of satisfactory and adequate highways and city streets. At the present time there are many competent paving engineers and asphalt technologists who consider the Hubbard-Field machine one of the best for designing sheet asphalt pavements

Mr. Prevost Hubbard is recognized as a leading authority on asphalt paving and has prepared many papers on the principles of design and the application of the Hubbard-Field machine to the design of asphalt pavements.

Marshall Machine—The investigational work which lead to development of the Marshall machine was initiated by the Corps of Engineers, Department of the Army, the later part of 1943 The work was directed toward the development of criteria to be used in constructing asphalt pavements suitable for various types of military airplanes

The criteria established for adequate pavements were based on results of traffic tests on actual pavements using properly weighted airplane wheel assemblies.

The laboratory and field work on which the criteria are based is contained in a Corps of Engineers report[5] published in 1948.

The established criteria for asphaltic concrete pavements adequate for traffic of heavy airplanes consist of numerical values for; Stability, Flow, Unit weight, Percent voids, and Percent voids filled with asphalt. The first two properties are measured by the Marshall machine, the remainder are calculated from values determined by standard laboratory tests. The criteria also include requirements for gradation and quality of aggregate and limits for percentage of mineral filler to be used in an asphaltic concrete pavement.

It was also determined from the investigation that a pavement conforming to the established criteria and having a Marshall stability of at least 500 lb. was satisfactory for the range of heavy airplane wheel loads used in the traffic tests

The investigational work indicates that the stability values of asphaltic concrete pavements designed in accordance with the es-

[5] Corps of Engineers report, "Investigation of the Design and Control of Asphalt Paving Mixtures" in 3 Vol., dated May 1948

tablished criteria are in general considerably higher than the minimum 500-lb. Marshall stability required

The Corps of Engineers report[5] contains the supporting laboratory and field data on which the pavement criteria were established. The Marshall machine and established pavement criteria have been used successfully by engineering agencies other than the Corps of Engineers not only for design of bituminous pavements but for control, preparation, and laying of mixtures in the field

The Corps of Engineers is continuing investigational work on bituminous paving mixtures to provide pavements adequate for the traffic of all types of newly developed military airplanes.

Hveem Stabilometer—The development of the "Stabilometer" was started about 1930 by F. N. Hveem[6], Staff Materials and Research Engineer, California Division of Highways, Sacramento, California. Many improvements were subsequently incorporated in the machine as the result of additional investigational work.

The correlation of Stabilometer test results with the performance of bituminous pavements under traffic is described in the following quotation from a preprint of Mr Hveem's paper[7] prepared for ASTM 1949 meeting in San Francisco, California: "The test has been in use for more than 15 years and a large mass of data exists to demonstrate the correlation between test results and performance of soils, base materials and bituminous surfaces under motor vehicle traffic."

The practicability of the application of the Stabilometer to construction of adequate bituminous pavements is evidenced by the following references contained in the State of California Division of Highways Standard Specifications, dated January 1949.

1. Page 32, Par. (9), "Stabilometer Test." This paragraph describes the test.

2. Page 173, Section 31, Asphaltic Concrete Pavement, Par. (d)(5) "Stability" This paragraph specifies the required minimum stabilometer value for an asphaltic concrete pavement.

As the test has been in use for at least 15 yrs. and the mileage on California primary highway system is nearly 14,000 mi., it seems reasonable to conclude that a considerable mileage of bituminous pavements was designed on the basis of stabilometer tests.

ANALYSIS OF FIGURE 31 DEMONSTRATING THE INADEQUACY OF AN UNCONFINED COMPRESSION TEST FOR MEASURING THE STABILITY OF BITUMINOUS PAVING MIXTURES

Dr. McLeod only infers that Figure 31 is applicable to the Marshall machine. However, in his comments on a paper[8] by Mr. John M. Griffith, Corps of Engineers, a reproduction of Figure 31, with some few changes in symbols, designated as Figure A is used by Dr. McLeod. The following is a quotation in connection with Figure A: "The Marshall test is quite unsuited to measure the stability of bituminous mixture because it is essentially an unconfined compression test The fundamental inadequacy of such a test for measuring the stability of bituminous mixtures can be quite easily demonstrated. Figure A is employed for this purpose."

As Dr. McLeod's Figure 31 is a reproduction of Figure A mentioned in the foregoing quotation it is assumed that the Marshall test is being referred to directly.

A number of assumptions regarding Marshall values have been introduced in Figure 31. However, only those assumptions considered to be particularly pertinent to the Marshall test are discussed.

Two of the assumptions introduced in Figure 31 are:

1. Three different mixtures represented by Mohr envelopes xw, yv, and zu in Figure 31a have the same unconfined compressive strength (Marshall stability value) for the corresponding value for c and ϕ indicated. Further, mixtures represented by envelopes yv and zu would be unstable for the vertical load indicated

2. Three different mixtures represented by Mohr envelopes lr, mq and np in Figure 31b have three different unconfined compressive

[6] Member of the Triaxial Institute

[7] Preprint, F N Hveem, "Application of the Triaxial Test to Bituminous Mixtures—Hveem Stabilometer Method," ASTM, San Francisco, Calif , October (1949)

[8] John M Griffith, "Evaluation of the Method of Asphalt Pavement Design Developed by the Corps of Engineers," *Proceedings* AAPT, 1949

strengths (Marshall stability value) for the corresponding values of c and ϕ indicated.

Further, mixtures represented by envelopes lr and mq would be rejected by the Marshall test as unstable for the vertical load indicated although they would prove entirely adequate based on the triaxial values of c and ϕ shown.

A third assumption is made, "Consequently, it is apparent from Figure 31, the unconfined compression test (Marshall stability value)[9] either may or may not be influenced by wide changes in the composition of bituminous mixtures, which are reflected by large variations in the c and ϕ values."

As in the case of the assumptions used in the equations and other figures, these assumptions are not supported by any comparative laboratory data showing the actual Marshall values of the mixtures which have the triaxial values of c and ϕ indicated.

A digest of the investigational report[4] will disclose that the Marshall machine adequately reflects and properly measures the pertinent variations which are normally found to exist in asphaltic concrete pavements. The variations are as follows:

1. Size, angularity and type of coarse aggregate
2. Percentage of coarse aggregate
3. Percentage and penetration of asphalt
4. Percentage and character of mineral filler
5. Angularity of fine aggregate
6. Gradation of aggregate (both fine and coarse)
7. Mixing temperatures.

The Marshall stability machine reflects the variations of a paving mixture in a similar manner as to the values for c and ϕ in the triaxial test.

In view of the above, it is highly improbable that (1) paving mixtures represented by Mohr envelopes xw, yu and zu with the corresponding values for c and ϕ indicated in Figure 31a would have the same Marshall stability value or (2) paving mixtures represented by the Mohr envelopes lr, mq, and np with the corresponding values for c and ϕ indicated would have the three different Marshall stability values assumed

Until Dr McLeod submits the composition of the paving mixtures and both the Marshall and triaxial values for each mixture used in Figure 31, it must be considered in connection with the first two assumptions that:

1. The inadequacy of the Marshall machine is not proven by Figure 31.

2. Figure 31 is purely a theoretical chart in which assumptions are used as a basis to obtain conclusions which may not be valid or technically correct.

The third assumption that it is apparent from Figure 31 that the Marshall test may or may not be influenced by changes in the composition of the mixtures is not considered valid for at least two unsupported assumptions previously mentioned have been introduced in Figure 31. Further, the failure of the Marshall machine to measure properly the changes in composition of a paving mixture is not borne out by the investigation of asphalt paving mixtures conducted by the Corps of Engineers.

[9] "Marshall stability value" has been inserted by the author.

SUMMARY AND CONCLUSIONS

1. It cannot be doubted that a practical method for the design of bituminous pavements based on triaxial testing adequately correlated with field conditions will ultimately be developed. Neither can it be doubted that the Corps of Engineers and other engineering organizations will give due consideration to a design method developed by the Triaxial Institute as its personnel is composed of able investigators, competent highway engineers, university professors and representatives of the asphalt industry thoroughly experienced in pavement design. However the Triaxial Institute recognizes the need for further investigational work on the items listed below and a program for the accomplishment of the necessary work has been initiated.

a Improvement in preparation of test specimens of paving mixtures so they will have physical characteristics comparable to finished pavements.

b. Adequate correlation of paving mixtures designed by the triaxial test with performance under traffic.

c. Interpretation of the triaxial method of design into simple terms suitable for use by the practicing engineer and laboratory technician.

d. Correlation of the triaxial test with other tests currently being used for design of bituminous pavement.

2. Numerous assumptions unsupported by

engineering data have been introduced into the equations and figures presented as the basis for Dr. McLeod's rational design of bituminous paving mixtures.

3 Past history indicates that the Hubbard-Field, Marshall, and Hveem testing machines and methods have been used successfully by engineers for a considerable period of time in designing adequate bituminous pavements for highways, city streets, and airfields. Dr. McLeod's criticism that these testing machines and methods are inadequate, misleading and confusing is unwarranted and utterly without foundation based on experience in the use of these machines by experienced design engineers.

4. Dr. McLeod fails to furnish the comparative laboratory Marshall and triaxial values of the paving mixture used in Figure 31 on which an attempt is made to prove the inadequacy of the Marshall machine for measuring the stability of bituminous paving mixtures.

5 It is considered that before any method, either empirical or theoretical, can be used with confidence for designing economical asphalt pavements adequate for heavy traffic, correlation of the effect of actual traffic on pavements designed by the method is necessary.

In view of the foregoing comments the description of the method for rational design of bituminous paving mixtures proposed by Dr. McLeod must be considered only a highly theoretical and academic discussion, the validity of which is questioned at this time.

Norman W. McLeod, *Closure*—The discussion presented by Mr. Ricketts is welcomed and appreciated, since between his comments and our reply a better understanding of the objectives and subject matter of our paper should result. Incidentally, Mr. Ricketts' remarks serve to support the stormy weather forecast made in the final sentence under the heading "Conclusion" in our paper, which states, "the development and general introduction of a rational method of design for bituminous paving mixtures will probably be just as controversial a topic during the next few years as the problem of the overall thickness of flexible pavements has proven to be."

In the first part of his paper Mr. Ricketts lists two principal reasons for presenting his discussion, the first of which is,

"The possibility that the casual reader may infer that Dr McLeod's rational method for design of bituminous pavements is now ready for immediate use by the design and construction engineer and the numerous charts and equations are technically correct, fully supported by engineering data, and adequately correlated with pavement performance in the field."

The author believes that adequate precautions were taken in the paper itself to avoid this possibility With several exceptions, the diagrams included in our paper are not intended to be final design charts Their principal purpose is to illustrate graphically the influence that various important variables, such as K, n, f, g, P, Q, t, etc., may have on the stability of a bituminous pavement in the field. It is clearly recognized that not enough is known about the actual magnitude of some of these variables at the present time to make adequate or maximum use of them in the design of bituminous pavements. Considerable laboratory and field investigation will be necessary to provide average values and the range of values possible for these different factors. In the meantime, the mathematical equations including these variables can be derived and, by means of graphs based upon these equations, the influence of different values for each factor can be usefully demonstrated.

Until more is known about the actual influence of these variables on bituminous pavement design, sound engineering requires that they be employed very conservatively. It was with this in mind that the last paragraph, under the sub-heading CRITICAL CONDITIONS FOR DESIGN, was included in our paper. This paragraph reads as follows:—

"Finally, until there has been an opportunity to build up information on the field performance of bituminous mixtures designed by the rational method, and employing the triaxial test, it would seem prudent to base the design of bituminous mixtures on the stationary load condition, for the maximum pressure applied to the contact area (*14, 15*), and possibly including an impact factor (*23*). *If particular care is taken to obtain a strong bond between pavement and base*, it is believed that the curves of Figure 14, for which K = 1, would provide a conservative basis for design. Later, as confidence in this method of design becomes established, and as more experimental data become available, the refinements indicated in the paper, particularly with regard to Figures 17, 27, and 30, which might lead to a less conservative design for both static and moving loads, could be gradually adopted."

It is believed that this paragraph quite adequately answers the criticism made by Mr. Ricketts in this respect.

The author happens to lack the laboratory equipment and personnel required to obtain the considerable data needed for the complete documentation of every step in the development it presents. Nevertheless, we entirely disagree with the statement appearing in Mr. Ricketts' closing comments, "the description of the method for rational design of bituminous paving mixtures proposed by Dr McLeod must be considered only a highly theoretical and academic discussion." Does Mr. Ricketts suggest, for example, that the remarkable and experimentally proven influence of the rate of strain on the stability measured for a bituminous mixture, demonstrated in Figures 18 and 19, is "theoretical and academic"? Few properties of bituminous mixtures are of more practical significance in providing a fundamental understanding of the variations in their performance under different traffic conditions.

As the paper itself clearly indicates, the development presented is based essentially on the Mohr diagram, the Coulomb equation, the influence of rates of strain or rates of loading, and the balancing of applied stresses and resistances. The last of these is simply a problem in engineering mechanics. The Mohr diagram and Coulomb equation have been employed as highly useful tools for a considerable number of years to obtain practical solutions to soil mechanics' problems. The stability of bituminous pavements appears to be a special problem in soil mechanics. Consequently, we cannot follow the line of reasoning that leads Mr. Ricketts to conclude that principles, recognized to be of the greatest practical value for the solution of stability problems in the field of soil mechanics, should suddenly become "theoretical and academic", when utilized to solve the problem of the stability of bituminous pavements.

In connection with our Figure 11, Mr. Ricketts states, "Here again is an assumption, as no engineering data are offered to substantiate the fact that all mixtures considered satisfactory possess an L value of 30 psi." Figure 11 is simply the graphical solution obtained when equation (3) is employed to obtain the answer to a specific problem for which the stress conditions were clearly defined, i.e. $V = 100$ psi, $L = 50$ psi. Figure 11 is intended to apply only to the material *within* an element carrying these specified stresses under equilibrium conditions. Figure 11 is not concerned with the nature of the material *outside* the stressed element, except that it must be placed in such a manner as to provide a lateral support L of 30 psi. Consequently, Figure 11 is not intended to deal with the question raised by Mr. Ricketts concerning how much lateral support the material *outside* the element can provide. This problem, however, is taken up in connection with Figures 12, 13, and 14

The text of the paper itself clearly points out that equation (3) and design diagrams like Figure 11 are of little practical value unless some means can be developed for determining the amount of lateral support L provided by the pavement adjacent to the loaded area. A method for this purpose is described in connection with Figures 12, 13, and 14. This method indicates that the unconfined compressive strength of the paving mixture, multiplied by a factor K, provides a reasonable evaluation of this lateral support L. Figure 17 demonstrates the marked influence that different values of K would have on bituminous mixture design. No experimental data concerning the magnitude of the value of this factor K for bituminous pavements in service seem to be available. Text books on soil mechanics, however, show indirectly that a value for $K = 1$ should ordinarily be conservative. Consequently, any engineer wishing to use the method of design outlined would probably be quite safe in assuming a value for $K = 1$. That is, the assumption that the lateral support L available is equal to the unconfined compressive strength would appear to be conservative.

It is not improbable that the structure of a bituminous pavement is somewhat different in a horizontal than in a vertical direction. If this should be the case, and is of sufficient importance, the value of the unconfined compressive strength to be employed for evaluating L might have to be measured by

Haefeli's procedure for running the triaxial test, in which the vertical load V becomes the minor principal stress, and the horizontal pressure L is the major principal stress.

Mr. Ricketts states that values for f, the coefficient of friction between pavement and tire, and for g, the coefficient of friction between pavement and base, have not been furnished. This statement is not correct concerning the factor f, since reference is made in the paper to the experimental values obtained by Moyer and by Giles and Lee. No values for g seem to be available.

In the absence of any recorded values for g, it was not possible for this particular paper to do more than indicate the important influence that different values for f and g would appear to have on any rational design of bituminous paving mixtures. As in connection with the factor K, the several diagrams illustrating the effect of different values of f and g on pavement stability indicate the desirability of carrying out the necessary experimental work to evaluate these factors precisely for different paving mixtures and base course surfaces, and possibly for different tires. In the meantime, for purposes of design, it would be advisable to employ conservative values for f and g.

It is not clear to us why Mr. Ricketts, in his opening paragraph, attempts to create the impression that there is some close connection between the subject matter of our paper and that of two papers by V. R. Smith, and that the triaxial equipment and test procedure outlined in our paper is similar to that recommended by Mr. Smith The latter has very clearly indicated that his method of dituminous mixture design is based upon the theory of elasticity. This is not true of the subject matter for the author's paper. Smith recommends the testing of bituminous mixtures at zero rate of strain, since this avoids the viscous resistance factor. Our paper recommends that the rate of strain adopted should bear some relationship to the nature of the stresses imposed by traffic and may, therefore, be variable. Reference to the influence of pavement thickness, of frictional resistance between pavement and tire and between pavement and base, of the lateral support of the pavement surrounding the loaded area, etc , on pavement stability is made in our paper, but not in those by Smith. To anyone who makes even a casual study of the papers by Smith and by the author, it must be perfectly clear that the approaches adopted by each are fundamentally different. In pointing out and emphasizing this fact, no criticism of Mr. Smith's method is even implied here. He has done some excellent work with the triaxial test Nevertheless, it should be clearly understood that the approach described by Mr. Smith is entirely different from our own.

Mr. Ricketts reviews briefly the objectives of the Triaxial Institute. We can only add "Amen" to the comments he has made on the highly commendable program the Triaxial Institute has laid out for itself. In item (2), under the heading GENERAL in our paper, reference is made to the particularly meritorious work of this Institute towards the development of a compaction procedure that will provide test samples with the same structure that develops under rolling and traffic in the field.

Mr. Ricketts places special emphasis on the Triaxial Institute's realization that test specimens duplicating pavement structure in the field are necessary before a sound method of design based upon the triaxial test can be established We are in whole-hearted agreement with this, and have expressed the same sentiment in our paper.

We would have been more impressed with Mr. Ricketts' comments in this connection, if he had somewhere in his discussion stated with equal emphasis that this is also true of the various empirical tests, such as Hubbard-Field, Marshall and Hveem Stabilometer, in common use at this time. Surely the development of a method for preparing test specimens that will duplicate pavement structure in the field is just as important in connection with the use of these empirical tests, as in the case of the triaxial test, and particularly since the results of these laboratory tests must be tied in with field performance, if the tests themselves are to be of any practical value.

Mr. Ricketts refers to our Figure 31, which demonstrates the fundamental inadequacy of the unconfined compression test for measuring the stability of bituminous paving mixtures. Insofar as the Marshall stability test is related to the unconfined compression test, the remarks in our paper with reference to Figure 31, apply to it also.

Mr. Ricketts states "The Marshall stability machine reflects the variations of a paving mixture in a similar manner as to the values for c and ϕ in the triaxial test" A moment's reflection should indicate that this statement cannot be true Graphs of Marshall stability values versus bitumen content shown in Highway Research Board Research Report 7-B, "Symposium on Asphalt Paving Mixtures", indicate that, except at the peak, there are two points on each curve where the Marshall stability value is exactly the same, but the asphalt contents are different, one being on the lean side and the other on the rich side of optimum. It would be quite unusual for these lean and rich mixes to have identical values for c and ϕ in a triaxial test. This provides a very simple example, therefore, of variations in the composition of a paving mixture that do not change the Marshall stability value, but would result in different Mohr envelopes as shown in Figure 31(a).

On page 246, Volume 18, *Proceedings* of The Association of Asphalt Paving Technologists, Vokac, by means of an isometric chart, indicates the wide range of changes that can be made in the composition of bituminous mixtures and yet maintain a constant value for the unconfined compressive strength. Rice and Goetz provide similar diagrams in the same volume No one familiar with triaxial testing would suggest that all of these mixtures of widely varying composition, but with a constant value for unconfined compressive strength, could be represented by a single Mohr envelope, that is, by constant values for c and ϕ. Consequently, although no actual test data are provided in connection with Figure 31, the information obtained by other investigators strongly infers that the relationships shown in Figure 31 between unconfined compression and triaxial tests are correct. Insofar as the Marshall test is related to the unconfined compression test, the relationships indicated in Figure 31 apply to it also.

Mr. Ricketts objects to our advocacy of the triaxial test, because he believes that the work of the Triaxial Institute indicates it to be in only the development stage. Can it be safely stated of the three empirical tests in most common use, Hubbard-Field, Marshall, and Hveem Stabilometer, that they are completely out of the development stage? In connection with the use of the Marshall test, Mr Ricketts states that the Corps of Engineers is still "continuing investigational work on bituminous paving mixtures to provide pavements adequate for the traffic of all types of newly developed military airplanes." If wheel load or tire pressure were changed radically, it would be found that further development work was required in connection with the Hveem Stabilometer and Hubbard-Field tests also.

It should be observed that The Asphalt Institute recommends the design of asphaltic concrete by means of the triaxial test, in its "Manual on Hot-Mix Asphaltic Concrete Paving." While it was established on a much different basis than that outlined in our paper, and should be eventually modified, the author believes that the triaxial method described by The Asphalt Institute, together with the stability diagram accompanying it, is conservative, and can be safely used for asphalt pavement design for highways and city streets, *provided the pavement is firmly bonded to the base course.*

Mr. Ricketts writes that "Dr. McLeod's criticism that these testing machines and methods (Hubbard-Field, Marshall, and Hveem) are inadequate, misleading and confusing, is unwarranted and utterly without foundation based on experience in the use of these machines by experienced design engineers." While Mr Ricketts' version of the author's criticism of these three tests is much more strongly worded than he will find in any single sentence in the paper itself, nevertheless, after careful study of this matter, the author is convinced that the Hubbard-Field, Marshall and Hveem Stabilometer tests are fundamentally incapable of providing a satisfactory measure of the stability of bituminous pavements in service. The author is also convinced that of all the stability tests with which he is familiar at the present time, the triaxial is the only practical test which is fundamentally capable of providing data for this purpose.

The principal reason for the author's conviction in this respect is the simple observation that *the stability of a bituminous pavement in service may be quite different from that indicated by the results of an empirical stability test made on the paving mixture in a laboratory* The empirical tests are incapable of providing information to explain this fact, while as shown in our paper, the triaxial test does

Every engineer having wide experience with bituminous pavements has observed paving mixtures develop instability in the field, although high stability values had been or would be reported for them in the laboratory. Conversely, he has seen mixtures, for which laboratory tests would show questionable stability, perform without developing any indications of instability in the field. Thus practical experience has demonstrated the truth of the statement in the previous paragraph, that the stability value reported for a bituminous mixture by an empirical test in the laboratory does not necessarily represent the stability that the same mixture will develop after being laid as a pavement in the field.

As pointed out in the paper, all other factors being equal, the stability of a bituminous pavement in service may depend upon:—

1. Its thickness
2. The frictional resistance between pavement and tire
3. The frictional resistance between pavement and base
4. Whether the applied wheel load is stationary, or is moving at a relatively uniform rate of speed, or is subjecting the pavement to severe braking and acceleration stresses.

The three empirical tests, Hubbard-Field, Marshall and Hveem Stabilometer, are fundamentally incapable of explaining why any one of the four factors just outlined should have an influence on the stability that a paving mixture may develop under traffic when laid as a pavement. As indicated in the paper under discussion, the triaxial test provides fundamental information that makes it a reasonably simple matter to understand why these four factors may have a very marked influence on the stability that a paving mixture will develop in the field.

The influence of thickness on pavement stability was referred to in the paper itself, but can probably be more easily understood by reference to the accompanying Figures A, B and C. Figures A and B illustrate the influence of thickness on pavement stability under a stationary wheel load, or under a wheel load moving at a uniform speed. Figure C demonstrates the effect of thickness on pavement stability, when the pavement is subjected to severe braking or acceleration stresses.

In Figure A the principal curved line represents the pressure ordinates across the trans-

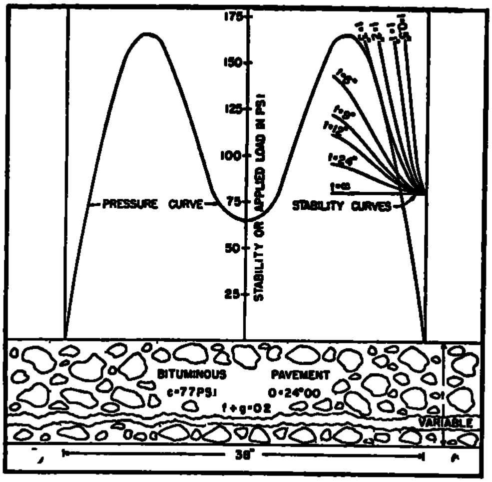

Figure A. Influence of Pavement Thickness on Pavement Stability for Stationary Wheel Loads, or Wheel Loads Moving at a Uniform Speed (Airplane Tire)

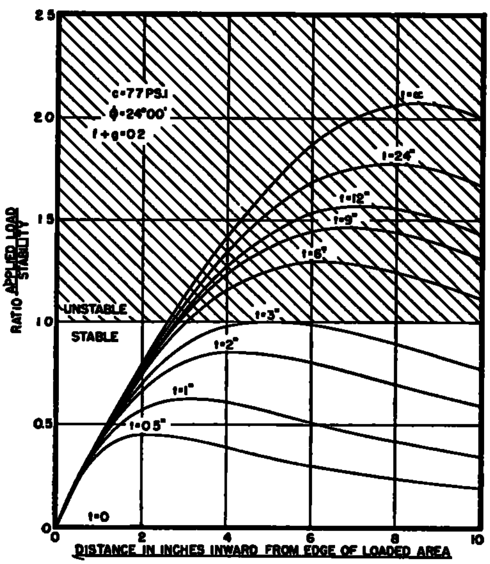

Figure B. Influence of Pavement Thickness on Pavement Stability for Stationary Wheel Loads or Wheel Loads Moving at a Uniform Speed (Airplane Tire)

verse axis of the contact area of a large airplane tire. This pressure curve was con-

GENERAL EQUATION

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}\;2K\tan\phi\frac{1}{w} + J + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + \frac{1}{w}}{\frac{1-\sin\phi}{1+\sin\phi} + \frac{1}{t}(f-g)}\right]$$

WHEN $f-g = 0.25$, $K=1$, $J=1$ AND $\frac{1}{w} = 1.5$, THE GENERAL EQUATION BECOMES

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}\,(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 0.25\frac{1}{t}}\right]$$

WHEN $\frac{1}{t} = 0$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi}}\right]$$

WHEN $\frac{1}{t} = 0.5$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 0.125}\right]$$

WHEN $\frac{1}{t} = 1$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 0.25}\right]$$

WHEN $\frac{1}{t} = 2$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 0.5}\right]$$

WHEN $\frac{1}{t} = 4$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 1}\right]$$

WHEN $\frac{1}{t} = 10$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 2.5}\right]$$

WHEN $\frac{1}{t} = 20$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 5}\right]$$

WHEN $\frac{1}{t} = 50$

$$V = 2c\left[\frac{\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}(3\tan\phi+1) + \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} + 1.5}{\frac{1-\sin\phi}{1+\sin\phi} + 12.5}\right]$$

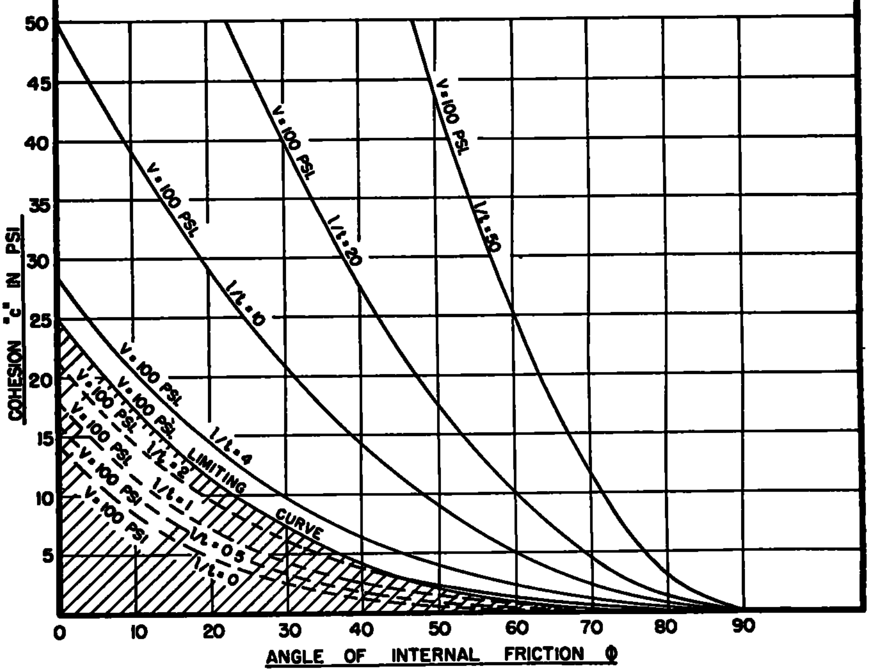

Figure C. Influence of Thickness on the Design of Bituminous Pavements Subject to Severe Braking Stresses, for Values of f-g.

structed by smoothing out the test data for the pressure distribution across the contact area for a wheel load of 200,000 lb., that appears in Plate 92 of O. J. Porter's report on Stockton Test No. 2. The stability curves shown in the upper right hand side for different thicknesses of a given bituminous paving mixture ($c = 7.7$ psi, $\phi = 24$ deg) were determined by a modification of equation (15),

$$V = 2cK \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi} \frac{1 + \sin \phi}{1 - \sin \phi}} + 2c \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} + nV''(f + g) \left(\tan\left(45 - \frac{\phi}{2}\right)\right) \frac{1 + \sin \phi}{1 - \sin \phi} \qquad (27)$$

where

- V = the stability developed by the bituminous pavement at any point on the contact area
- V'' = the average unit vertical pressure over the distance between the edge of the loaded area and any point on the contact area for which the stability value V is required

and the other symbols have the significance previously defined for them.

For the stability curves shown in figure A, the values of $K = 1$ and $f + g = 0.2$ were assumed for purposes of illustration. The thickness t pertaining to the stability curves can be obtained by reference to equation (11) and Figures 21 and 26.

If adequate design requires that the pavement should develop sufficient stability that no part of the contact area is overloaded (that is, the stability developed is to be not less than the applied load at any point on the loaded area), figure A demonstrates that the maximum thickness of pavement that can be used for the particular conditions pertaining to this diagram, is 3 in. The stability curve for a thickness of 6 in, for example, cuts through the curve representing applied pressure, and indicates that for this thickness the applied pressure would be greater than the pavement stability over a considerable portion of the contact area Figure B is based upon figure A and illustrates the relationship between applied load and stability at different distances inward from the edge of the contact area, for different pavement thicknesses, in somewhat different form. It also demonstrates that for the particular conditions pertaining to figure A, pavement instability can be expected over some portion of the contact area, as soon as the pavement thickness becomes greater than 3 in. The pavement is obviously overloaded at any point on the contact area where the ratio of applied load versus stability is greater than unity.

Therefore, Figures A and B indicate that increased pavement thickness tends to be a liability as far as pavement stability is concerned, when designing for stationary wheel loads, or for wheel loads moving at a uniform rate of speed. That is, to support a given vertical load V under these traffic conditions, a thick pavement must be designed to have a higher minimum stability (higher c and ϕ values) than a thin pavement

Figure C demonstrates the influence of the ratio of l/t on the design of a bituminous pavement to be subjected to braking stresses, when $f - g = 0.25$. The symbol l refers to the length of the contact area, Figure 29, and t represents pavement thickness. It is apparent from Figure C that with all other factors equal the higher the value of the ratio of l/t, the more stable must the paving mixture be (higher values for c and ϕ), when the pavement is subjected to severe braking stresses under a given vertical load V. A higher value for the ratio l/t is obtained either by keeping l constant and decreasing the pavement thickness t, or by maintaining the thickness t constant and increasing the length l of the contact area.

When $f - g$ is positive, Figure C demonstrates that to resist a given braking stress, a thin pavement must be designed to have a higher minimum stability (higher values for c and ϕ), than a thick pavement. A study of Figure 29 demonstrates that this latter conclusion is to be expected, since the resistance to braking stresses increases with an increase in the area of the sides abhg and dcij. For a given length l of contact area, it is apparent that the area of these two sides will increase when thickness t is increased, and vice versa.

It should be particularly noted that the conclusions just expressed in connection with Figure C, concerning the influence of pavement thickness on pavement stability, apply to the design of bituminous pavements that are to be subjected to severe braking stresses,only when the value for $f - g$ is positive. From the general equation at the top of Figure C, it can be seen that when designing for severe braking stresses, pavement thickness has no influence on pavement stability when $f - g = 0$, and that pavement stability decreases with an increase in pavement thickness, when $f - g$ has negative values. However, as pointed out in connection with Figure 30, when designing for severe braking stresses, it appears that when $f - g = 0$, or has negative values, the tendency of the vertical load V to squeeze the pavement out from under the tire presents a more critical condition of design than the braking stress itself.

The conclusions from Figures A, B, and C, concerning the influence of pavement thickness on pavement stability, are in keeping with practical observation and experience, but could be neither derived nor explained on the basis of any test data provided by the Hubbard-Field, Marshall, or Hveem tests.

Mr. Ricketts refers to the fact that a considerable mileage of pavements for highways streets and airports have been designed by Hubbard-Field, Marshall and Hveem tests, and that good correlation between test methods and field performance has been established. One is inclined to enquire just how carefully and thoroughly this correlation has been studied and what price is being paid in terms of unnecessary overdesign where such correlation has been demonstrated. There is very good reason for suspecting that where good correlation has been established, either with or without excessive overdesign, it is not entirely due to the merit of the empirical stability test employed, but is partly due to the assistance unwittingly given by the construction engineer on the job, through his care in providing a good bond between pavement and base. There is nothing in the design data obtained from the Hubbard-Field, Marshall or Hveem tests to indicate that the stability of the finished pavement may depend very materially on good frictional resistance between pavement and base. Nevertheless, construction engineers have instinctively made provision for good bonding between pavement and base through prime coat, tack coat, etc We suspect that in locations where this bonding of pavement to base is weak or indifferent, the correlation between the stability values provided by these empirical tests and the field performance leaves considerable to be desired.

The triaxial test, on the other hand, provides data from which, as shown in the paper, the important effect of a good bond between pavement and base on pavement stability can be clearly demonstrated The clear recognition of the importance of this fact could lead to considerable economy in bituminous pavement design and construction.

A serious disadvantage of the use of empirical tests is that the actual degree of overdesign or underdesign of a bituminous pavement is never known. In most engineering fields safety factors are used, but from the laws of the strength of the materials employed, a reasonable estimate of the safety factor can be made. This cannot be done in connection with our bituminous pavements as long as we continue to rely on the present empirical tests The development of a rational method of design would give some control over the safety factor employed This in turn should lead to worthwhile pavement economy, because of the greater confidence in local materials, and the wider selection of aggregates it should make possible.

It should be pointed out that the author is not alone in his criticism of our currently used empirical tests. A concise review of the shortcomings of empirical tests is contained in Nijboer's book "Plasticity as a Factor in the Design of Dense Bituminous Road Carpets." Smith has also criticized some of our present empirical tests in his paper for Vol. 18 of the AAPT *Proceedings* and in his discussion in Highway Research Board Research Report 7-B, "Symposium on Asphalt Paving Mixtures"

The art of designing bituminous pavements is still ahead of the science, and it is inevitable that empirical tests should have been developed during the past in an endeavour to measure stability. However, we have now reached the stage where serious efforts are being made to emancipate the design of bituminous pavements from empirical tests and to establish it on a sound rational basis. Endersby, Smith, Nijboer and others have made important contributions in this direction. The writer's paper represents another attempt to break away from the previous beaten path. There will be others.

Whether the present empirical tests can be correlated with any rational method of design ultimately developed, or whether one or more entirely new tests will be required, is for the future to answer. Be that as it may, the author believes that the time has come when serious consideration should be given to developing a rational method of design and to breaking away from our present dependence on empirical tests. As long as we maintain these empirical tests as our only basis for design, whenever a wheel loading or tire pressure changes materially, we are faced with another laborious cycle of laboratory testing and field correlation. In this respect, our present standard approach to bituminous pavement design is not far removed from the engineering technique of the ancients, who had to load the bridge they had just completed to failure, to determine what weight it could carry.

DEPARTMENT OF MATERIALS AND CONSTRUCTION

C. H. Scholer, Chairman

PLANT STABILITY TEST FOR HOT-MIX ASPHALTIC CONCRETE MIXTURES

DR. TING YE CHU, *Research Associate, Iowa Engineering Experiment Station* AND M. G. SPANGLER, *Research Professor of Civil Engineering, Iowa State College*

SYNOPSIS

A new method of testing for the control of asphaltic concrete mixtures at hot-mix plants is suggested. The unique feature of this test is the rapidity of its operation. Time from the molding of the specimen to the end of testing is about ten minutes.

A single mold has been developed for both molding and testing which eliminates the need for extruding the specimen. In addition, an armored thermometer is used to measure the temperature of a specimen before testing It is not necessary, therefore, to keep a specimen in a hot water bath or constant temperature oven for a certain period as required in ordinary stability tests.

These two developments give a rapid field stability test to help the plant inspector keep better control over the asphaltic concrete mixture at hot-mix plants.

Stability tests have long been employed as valuable aids in the design of asphaltic concrete mixtures By making such tests, it is possible to select better materials and to find the optimum combination of the selected materials, so that mixtures of high quality can be expected. Common stability tests for such a purpose are the Hubbard-Field, the Hveem, and the Marshall test.

Although the design of mixture is a necessary step for securing a quality asphaltic concrete pavement, control of the mixture at producing plants is no less important. At asphaltic concrete plants, it is hardly possible to keep everything exactly the same as has been assumed in the design. Whenever deviations from the design condition occur, it is possible that the quality of the mixture may be impaired even if such deviations are within specification limits. It is the duty of a plant inspector to check the quality of mixtures produced and, if necessary, make proper adjustments to the mix-formula.

In the prevailing practice for asphaltic concrete plant inspection, samples of the mixture are sent by the inspector to a nearby laboratory, where they are reheated, molded, and tested The inspector cannot get test results from the laboratory until one or more days afterward In case the mixture is found to be unsatisfactory by the laboratory stability test, it may be too late for adjustment because a